ACI News Letter

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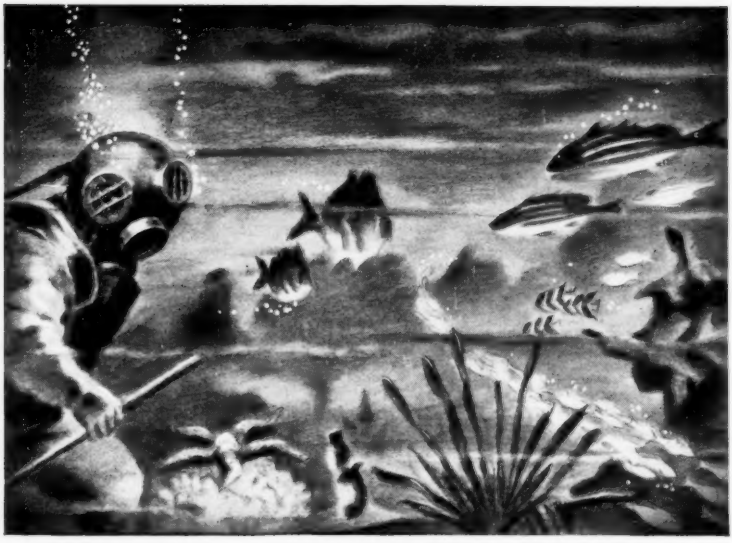

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ACI

News Letter Contents

January 1950

On the Cover—The Construction Practice Award to be presented by ACI this year is a bronze plaque 7¾ x 10½ in., mounted on a walnut base. It is given "for a paper of outstanding merit on concrete construction practice published in the Journal of the Institute."

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1949 ACI Awards

Richart, Siess, Newmark, Kelly, Schuman, Hornibrook and Asplund Honored

Frank E. Richart, C. P. Siess and N. M. Newmark, T. M. Kelly, L. Schuman and F. B. Hornibrook, and S. O. Asplund will receive ACI Awards from President Gilkey at the 46th Annual Convention luncheon at the Edgewater Beach Hotel, Chicago, February 21. These awards by the ACI Board of Direction were on the recommendations of the Awards Committee.

Frank E. Richart was the choice for the first Alfred E. Lindau Award

for his years of labor toward the improvement of reinforced concrete design practice.

This award was founded in 1947 by the Concrete Reinforcing Steel Institute in honor of the late Alfred E. Lindau, past president of the Institute, and is in the form of a bronze plaque bearing a bas-relief portrait of Mr. Lindau. It is given only for outstanding contributions to reinforced concrete design practice and is not mandatory each year.

C. P. Siess and N. M. Newmark will receive the Wason Medal for "the most meritorious paper" of the year,

Rational Analysis and Design of Two-Way Concrete Slabs

which appeared in the ACI JOURNAL, December, 1948.

T. M. Kelly, L. Schuman and F. B. Hornibrook will receive the Wason Medal for "noteworthy research" reported in

A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions

in the ACI JOURNAL, September, 1948.

Continued on p. 4

The Wason awards, established in 1917 by the late Leonard C. Wason, past president of ACI, consist of a bronze medal and certificate to each awardee. For material published in the latest volume of *Proceedings* of the American Concrete Institute an award is made for the "most meritorious paper." The research medal is given for original research work on a subject relevant to Institute activity and which within the year has been the subject of an Institute paper.

To S. O. Asplund will go the Construction Practice Award Plaque for his paper

Strengthening Bridge Slabs with Grouted Reinforcement

which was published in the ACI JOURNAL January, 1949.

This year, for the first time, the token of the Construction Practice Award is a plaque (see News Letter Cover). It was established in 1944 by the American Concrete Institute to recognize the man on the job for his resourcefulness in translating design into the completed structure.

The Award Winners

Frank E. Richart—Alfred E. Lindau Awardee

Frank E. Richart, ACI Member since 1917, has long been active in Institute administrative and technical affairs. He was president in 1939 and has served also on Program, Advisory, Publication and Technical Activities Committees, on the Board of Direction and as vice-president. He has been a member of the Standards Committee and is currently its chairman.

Author or co-author of 22 JOURNAL papers and reports, much of his work has been devoted to reinforced concrete column and footing investigations. He was Wason Medalist in 1938 for "Rapid and Long-Time Tests of Reinforced Concrete Knee Frames." From supervision of column tests at the University of Illinois he moved up to chairman of the Institute's committee on Reinforced Concrete Column Investigation and coordinated the work being done in this field at several universities.

He is research professor of engineering materials at the University of Illinois where he was graduated in civil engineering (B. S. 1914, M. S. 1918, D. E. 1922). From 1913 to 1917 he had varied experience in railway maintenance, valuation field work, reinforced concrete building design and in testing. Since 1916 he has been in the Department of Theoretical and Applied Mechanics teaching and conducting research on engineering materials.

Professor Richart is a registered structural engineer and has written many bulletins and articles on indeterminate structures, proportioning concrete by the mortar-voids method, tests of reinforced concrete beams, columns, slabs, walls and frames and related structures.

He has found time to be active also in A.S.T.M., A.S.C.E., the International Assn. of Testing Materials and the Western Society of Engineers. He is a member of Tau Beta Pi, Sigma Xi and Phi Kappa Phi honor societies.

C. P. Siess and N. M. Newmark—Wason Medalists

C. P. Sciss, research assistant professor of civil engineering at the University of Illinois, graduated in civil engineering at Louisiana State University and then worked as soils engineer in the testing laboratory of the Louisiana Highway Commission. In 1937 he became special research graduate assistant in theoretical and applied mechanics at the University of Illinois where he received his M. S. in 1939 and his Ph.D. in 1948.

After work with the Chicago Dept. of Subways and Superhighways and the New York Central Railroad he returned to the University of Illinois in 1941 as special research associate and special research assistant professor of theoretical and applied mechanics, and was appointed to his present position in 1949.

Chairman of ACI Committee 215, Fatigue of Concrete, and member of

Committee 321, Design of Reinforced Concrete Slabs, and Committee 115, Research, Professor Siess also is affiliated with A.S.T.M., Highway Research Board, A.S.C.E., S.E.S.A., Sigma Xi, Tau Beta Pi, Phi Kappa Phi, Omicron Delta Kappa and Phi Eta Sigma.

N. M. Newmark, research professor of civil engineering at Illinois, is also active in ACI work as a director, as member of the Technical Activities Committee and as chairman of Committee 321, Design of Reinforced Concrete Slabs.

He was graduated with special honors in civil engineering at Rutgers University in 1930, and then went to the University of Illinois as a research graduate assistant in civil engineering. Since then he has been continuously on the university staff, and was appointed to his present position in 1947. Concurrently he served as consultant for several army and navy units during the war years, and has also done consulting work in structural design and machine analysis.

Doctor Newmark has had a number of technical papers published on such topics as structural analysis and design (See "Structural and Economic Studies of Monolithic Concrete Walls for Dwellings," ACI JOURNAL, May-June 1935, *Proc.* V. 31, p. 478), bridge floors, theory of elasticity, soil mechanics and fatigue of metals. In addition to his ACI affiliation, he is a member of A.S.C.E., A.S.M.E., Institute of Aeronautical Sciences and International Assn. of Bridge and Structural Engineers. He is also a member of A.S.T.M., A.R.E.A., Society of Experimental Stress Analysis, Phi Beta Kappa, Tau Beta Pi, Sigma Xi and Chi Epsilon.

T. M. Kelly, L. Schuman and F. B. Hornibrook—Wason Medalists

T. M. Kelly, now with the operations division, Corps of Engineers, Portland, Ore., District, attended Union College at Schenectady, N. Y. He graduated in civil engineering in 1933 and for 3 years worked for various federal and municipal agencies on field surveying, street and sewer projects. During 1936 and 1937 he was a calculating engineer with the American Locomotive Co., Schenectady. In 1937 he joined the National Bureau of Standards as materials engineer in the masonry construction section. He served in a similar capacity successively in the cement reference laboratory and the concreting materials section until April, 1948, when he assumed his present duties.

While with the National Bureau of Standards, Mr. Kelly spent a 3-year military leave on active duty with the naval reserve as an aircraft overhaul and repair officer. This activity included a concentrated course in aeronautical engineering at California Institute of Technology and overseas duty on Guadalcanal and Green Island.

L. Schuman was born in Manchester, England, came to the United States in 1910 and attended school in Philadelphia. He received a B. A. degree from Temple University in 1925 and did graduate work in physical chemistry at Massachusetts Institute of Technology. Author of several research papers on properties of cement and concretes, including tensile and wear testing, he has been with the National Bureau of Standards since 1926. During World War II he was in the ordnance development division and assisted in development of the magnetic fluid clutch. Mr. Schuman is now employed as engineer, test section, organic and fibrous materials division of the Bureau.

He is a member of the American Jewish Congress and a former member of ACI and American Chemical Society.

F. B. Hornibrook attended Willamette University and the University of Oregon, graduated from the State College of Washington in 1930 and went at once to the National Bureau of Standards where he served for 16 years. He received his master's degree in physical chemistry from the University of Maryland in 1934.

While at the Bureau of Standards, Mr. Hornibrook worked on calorimetry of

cement, development of an apparatus for measuring the vibration of concrete during compaction, and the development of a sonic apparatus for measuring the dynamic modulus of elasticity of concrete. He was author or co-author of many technical reports on these developments, has been active in technical committee work of ACI and A.S.T.M. In 1946 he joined the Master Builders Co., Cleveland, Ohio, becoming director of research, a position which he left to join the Best Fertilizer Co., Oakland, Calif., in 1947.

S. O. Asplund—Construction Practice Award

S. O. Asplund, was graduated from the Royal Institute of Technology at Stockholm, Sweden, in 1924 with a degree of C. E.; in 1943 he received his D. Eng. at the same university on his treatise "On the Deflection Theory of Suspension Bridges." Doctor Asplund worked with the American Bridge Co., the North Carolina Highway Dept., and the Cleveland Terminal Assn. from 1925 to 1929. As a contractor in Sweden since 1931 he has designed and built several flexible-type suspension bridges with concrete decks as well as numerous concrete structures and conventional concrete and steel bridges. He is a lecturer on bridge design at the Royal Institute of Technology, Stockholm, and a member of the Swedish Academy of Engineering Sciences. In addition to ACI he is a member of the Swedish Concrete Assn., the International Assn. for Bridge and Structural Engineering and A.S.C.E. He is the author of about 30 engineering papers; "Strengthening Bridge Slabs with Grouted Reinforcement" was his first contribution to American technical literature.

ACI's HISTORY—to, especially, long-time ACI'ers

An extra ACI staff chore—one that is more prolific of interest in its details than of time to give it—is to gather from miscellaneous records, including those from the walls and mental pigeon holes of personal memory, some incidents and anecdotes of ACI.

These are rarer in the early meetings and activities of the American Concrete Institute and its predecessor, the National Association of Cement Users (1905-1913), than in later years. But, early or late, your possible contribution might be one that would otherwise be overlooked. Items would be especially welcome that have long- or occasionally short-time significance and which give meaning to a general cause, or color to an ACI personality—along this line please address Harvey Whipple personally at 18263 West McNichols Road, Detroit 19, Michigan, who will undertake the task of a brief ACI history.

Program preview, 46th Annual ACI Convention, Edgewater Beach Hotel, Chicago

February 20-22, 1950

MONDAY, FEBRUARY 20

9 a.m.—Ballroom Foyer—Convention registration begins—Members and nonmembers—Registration fee $1.00.

Technical committee meetings as scheduled.

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2 p.m.—Ballroom—Convention called to order by President Herbert J. Gilkey

INSPECTION—J. W. Kelly, Session Leader

The Inspector—Miles N. Clair, Thompson and Lichtner Co., Inc., Boston, Mass.

SYNOPSIS—The functions of the inspector include cooperation with the contractor in planning for good workmanship and progress, observation of construction operations and the recording of important facts relative to the work, and the enforcement of specifications and good construction practice. The qualifications for the inspector are discussed; he must be experienced, interested, be able to cooperate, be honest and have lots of common sense.

Inspection and Testing of Materials—Nicolaas T. F. Stadtfeld, Board of Water Supply, City of New York (The paper appears in the December JOURNAL.)

SYNOPSIS—This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and with every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns on the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

Inspection of Building Construction—Leonard E. Dunlap, Carr and Wright, Inc., Chicago

SYNOPSIS—From the viewpoint of an architect, inspection on a job involving architectural and structural concrete encompasses a number of factors. Accurate plans and specifications, testing and developing the concrete mix design, proper erecting of forms and placement of concrete, and systematic curing require close attention by a competent superintendent and qualified inspectors.

Inspection and Control of Concrete for Highway and Bridge Construction—H. W. Russell, Division of Highways, Illinois Dept. of Public Works and Buildings, Springfield, Ill.

SYNOPSIS—Inspection procedures and personnel assignments used by the Illinois Division of Highways for large, medium and small jobs are described. The importance of assurance that materials used are adequately tested and approved before shipment, determination of proper proportions by trial mixtures, control of air entrainment, correction of batch weights and proper preparation and testing of concrete specimens are emphasized. The inspection of ready-mixed concrete supplied to small jobs poses a problem still lacking an adequate solution.

Inspection of Mass and Related Concrete Construction—Lewis H. Tuthill, U. S. Bureau of Reclamation, Denver, Colo. (The paper appears in the January JOURNAL.)

SYNOPSIS—This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual activities of inspection and those associated with concrete control since both have the same objective—a serviceable and presentable structure. Details of requirements and procedure to this end are discussed.

The Contractor's Viewpoint on Inspection—Nomer Gray, Walter Kidde Constructors, Inc., and Donald C. Andrews, Turner Construction Co., N.Y.C.

SYNOPSIS—The interests of owner and contractor are compared and recommendations are made on specifications, personal qualifications of the inspector, and methods of minimizing friction with the contractor.

Inspection of Ready-Mixed Concrete—E. L. Howard, Pacific Coast Aggregates, Inc., San Francisco, Calif.

SYNOPSIS—The production of ready-mixed concrete to meet many different specifications poses difficult inspection problems involving the control of aggregate grading and rapid changes in mix design. The limitations of scales and air-entraining admixture dispensing equipment must be considered in directions given to plant operators. A consistency meter and its application to ready-mixed concrete are described.

Summary of Inspection Practice—R. B. Young and Wilfred Schnarr, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada, summarize the papers presented at the inspection session.

TUESDAY, FEBRUARY 21

Concurrent Sessions will be held morning and afternoon in the **Ballroom** and **East Lounge**

9:00 a.m.—Ballroom—Vice-President Harry F. Thomson, Chairman

ACI BUILDING CODE STUDIES—Chester L. Post, Session Leader

Deformed Reinforcing Bars—Raymond C. Reese, Consulting Engineer, Toledo, Ohio

SYNOPSIS—The development of reinforcing bars from the original plain round or square forms to the present improved deformed patterns is traced through the early work of Withey and Abrams down to the 1949 report of Clark's tests. Noting that A.S.T.M. has adopted a standard for deformed bars, the author emphasizes the

need for ACI Committee 318 to revise design specifications to take into account the characteristics of the improved bars.

Two-Way Slabs—Suggested Changes in the Code—C. P. Seiss and N. M. Newmark, University of Illinois, Urbana, Ill.

SYNOPSIS—A new design specification for two-way concrete floor slabs is suggested. It is intended to replace the methods currently contained in Section 709 of the ACI Building Code (ACI 318-47). This specification is based on analyses of continuous rectangular slabs carrying a uniformly distributed load. Account is taken of the continuity of the slab, of the torsional stiffness of the beams, and the deflection of the beams.

PROBLEMS COMMON TO COMMITTEES 208, Bond Stress; 318, Standard Building Code; 323, Prestressed Reinforced Concrete; 324, Precast Reinforced Concrete Structures and 711, Precast Floor Systems for Houses—F. N. Menefee, Session Leader

Spacing of Moment Bars in Precast Joists—F. N. Menefee, University of Michigan, Ann Arbor, Mich., and H. L. Kinnear, University of Virginia, Charlottesville, Va.

SYNOPSIS—The paper describes tests of precast joists whose bar spacing is less than that provided for in the ACI Code. Theoretical considerations and actual tests are given for joists with $\frac{3}{8}$-in. spacing between moment bars. For the first series of 14 joists there is no indication that the $\frac{3}{8}$-in. spacing between bars reduced the effectiveness of the joists up to their rated capacity.

Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members—A. Amirikian, Navy Dept., Bureau of Yards and Docks, Washington, D. C.

SYNOPSIS—The use of relatively small size aggregates and favorable conditions for quality control make it possible to place reinforcing bars in precast concrete work at closer spacings than those specified or required in conventionally poured-in-place concrete construction Better quality control and utilization of rich mixtures also make it possible to obtain corrosion protection of reinforcement with relatively thin covers. Recommendations are given for a new basis of specifying bar spacing and cover in the form of proposed specifications applicable to precast concrete work, together with a discussion of some of the considerations justifying the suggested specifications.

Extent and Acceptability of Cracking in Precast Concrete Framing Members—A. Amirikian

SYNOPSIS—One of the important questions arising in concrete prefabrication concerns the acceptability of cracked framing members. Owing to the absence of definite guides for inspection, the fabricator often is penalized by unreasonable rejections. As an aid in this matter, a proposal is made for an acceptability clause which would give practical definitions of cracks and specify limitations for acceptability. The proposed clause would eventually form a part of specifications for precast concrete work.

The Patent Status of Prestressed Concrete and the Relationship of Prestressed Concrete to Building Code—Curzon Dobell, Preload Corp., N.Y.C.

SYNOPSIS—Based on a study of more than 100 domestic and foreign patents and on correspondence with committees of engineers in England, Switzerland, France and Sweden the paper gives (1) a brief account of the more important groups of patents and their use, and (2) a synopsis of codes in preparation in the above countries and the problems to be overcome in drafting a prestressed code for domestic use.

9 a.m.—East Lounge—Vice-President F. H. Jackson, Chairman

STRUCTURAL DESIGN OF CONCRETE PAVEMENTS—L. W. Teller, Session Leader

Introduction—L. W. Teller, Bureau of Public Roads, Washington, D. C., will introduce the subject of pavement design.

Influence of Subgrades and Bases on Design of Rigid Pavements—Kenneth B. Woods, Purdue University, Lafayette, Ind. (The paper appears in the January JOURNAL.)

SYNOPSIS—This paper was developed from research data, published reports, and experiences gained by observing the performance of both rigid and flexible pavements—particularly as related to subgrade soil textures and the use of base courses. Structural failures of rigid pavements, caused by large concentrations of exceptionally heavy loads, indicate the need for an evaluation of subgrades and bases in determining the most economical design of rigid slabs.

Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations—in regard to position and soils—the use of base courses must be evaluated against the economy of using slabs of increased thickness, more reinforcing steel, or combinations of the two.

It is concluded that it is not entirely feasible, with the present state of knowledge, to standardize rigid pavement design. Rather, the available data indicate that design practices should be developed by regions in which the subgrade soil, availability and type of base course materials, climatic conditions and traffic volumes and loads are evaluated.

The Problem of Slab Dimensions—A. T. Goldbeck, National Crushed Stone Assn., Washington, D. C.

SYNOPSIS—The dimensions of concrete pavement slabs necessary for optimum results depend on and are influenced by a great many factors, economic as well as physical. The factors and their relationship to slab dimensions are tabulated and discussed. The factors discussed include strength, elastic and plastic properties, volume change, subgrades, construction conditions and slab thickness, length and width.

Structural Design of Pavement Joints—William Van Breemen, New Jersey State Highway Dept., Trenton, N. J., and E. A. Finney, Michigan State Highway Dept., Lansing, Mich.

SYNOPSIS—The basic fundamental conditions involved in the design and construction of joints is discussed. Various types of joints and load transfer devices are described, as well as various forms of structural failure of joints. Recommendations are given for the design, fabrication and installation of load-transfer devices, the construction and finishing of the concrete surrounding the joint assembly and the sealing of joints.

What Can Steel in Concrete Pavements Do?—Bengt F. Friberg, Granite City Steel Co., Granite City, Ill.

SYNOPSIS—The early progress in the use of reinforcement in pavements is summarized. Typical failures of reinforced concrete pavement are discussed and recent developments in highway and airport pavements are described.

LUNCHEON

12:30 p.m.—Marine Dining Room—President Gilkey presents ACI awards.

Report of Annual Election and Introduction of New Officers

Retiring President's Address

2:30 p.m.—Ballroom—President H. J. Gilkey, Chairman

PRESTRESSED, PRECAST and REINFORCED CONCRETE

Review of Prestressed Concrete Construction—Thor Germundsson, Portland Cement Assn., Chicago

SYNOPSIS—The basic concepts of prestressed concrete are reviewed followed by examples of the application of several systems which utilize prestreteching and poststretching of the reinforcement.

A Simple Mathematical Tool for the Solution of Difficult Design Problems—Alfred Parme, Portland Cement Assn., Chicago

SYNOPSIS—This paper deals with the application of finite differences to the solution of those structural problems in which the physical relationships are expressed as a differential equation. Essentially the technique employed consists of replacing the derivatives of the differential equation by its central difference equivalent. The problem is thus reduced to the simple task of solving a system of simultaneous linear algebraic equations. The numerical computation involved in the procedure is considerably reduced by two devices. First, the number of equations necessary to attain sufficient accuracy is reduced by an evaluation of the error introduced in substituting central differences for derivatives. Secondly, the solution of simultaneous equations is speeded by a systematic rapid tabulation of easily determined values.

The procedure is applied to the design of a sheet pile wall, elliptical dome and skewed bridge to illustrate the scope and simplicity of the method.

Precast Concrete Construction in Britain—P. G. Bowie and A. R. Collins, Cement and Concrete Assn., London, England

SYNOPSIS—Precast frames for airplane hangars, garages and farm buildings are discussed. Several systems using precast units in housing construction are described and illustrated. Prestressed precast railroad ties and transmission-line poles are mentioned briefly.

Precast Reinforced Concrete Structures—C. D. Wailes, Jr., C. D. Wailes Corp., Los Angeles, Calif.

SYNOPSIS — Interesting and novel things are being done with precast concrete in many parts of the world. In recent years the technical literature has included good coverage of the developments and uses of this growing type of construction. Accounts range from very theoretical, academic studies in research to completed methods by which sound and satisfactory structures have been constructed. It is not the purpose of this paper to survey these many and varied undertakings, but rather to focus attention on the practical application of precast construction in this country.

Corrosion Protection of Thin Precast Concrete Sections—D. H. Pletta, E. F. Massie and H. S. Robins, Virginia Polytechnic Institute, Blacksburg, Va.

SYNOPSIS—A new electrical resistance technique for measuring the rate of corrosion of steel reinforcing is described in this paper. The method employs a thin ribbon 0.008 in. x 0.25 in. as the resistance element embedded in 6 x 12 in. thin panels, and a Kelvin bridge sensitive to 0.0001 ohm. The data plotted in dimensionless parameter form enable the *half life* of concrete to be determined at a comparatively early age. The term half life is defined as the time required for the cross-sectional area of the reinforcement to decrease by one-half its original value due to corrosion. Six mixes, three water-cement ratios, four exposure conditions and three depths of cover were examined.

2:30 p.m.—East Lounge—Past President R. F. Blanks, Chairman

ADMIXTURES—W. T. Moran, Session Leader

Introduction—W. T. Moran, U. S. Bureau of Reclamation, Denver, Colo.

Use of Admixtures to Increase Resistance to Freezing and Thawing—F. H. Jackson, Bureau of Public Roads, Washington, D.C.

SYNOPSIS—Various admixtures are discussed briefly as an introduction to more detailed treatment of air-entraining materials. The relative merits of admixtures and interground agents are considered. Optimum ranges of air content for different structural uses are given with particular reference to pavements.

Use of Admixtures in Concrete Products—Bruce E. Foster, National Bureau of Standards, Washington, D. C.

SYNOPSIS—Accelerators air-entraining agents, gas-forming agents, water repellants, and workability agents are considered in connection with concrete products. Effects on the finished products and precautions in the use of additives are included.

Use of Admixtures in Correcting Grading Deficiences—T. C. Powers, Portland Cement Assn., Chicago

SYNOPSIS—The factors affecting bleeding characteristics and workability of fresh concrete are reviewed and the effect of admixtures on these properties is assessed.

Use of Admixtures in Counteracting Alkali-Aggregate Reactions—W. T. Moran

SYNOPSIS—A review of present knowledge of this subject. It is emphasized that further studies may revise thinking in this field.

Use of Admixtures as Integral Dampproofing Materials—Bruce E. Foster

SYNOPSIS—The mechanics of the passage of water through concrete and the effect of dampproofing materials on permeability are discussed. Such materials are not considered substitutes for good practice and workability.

GENERAL TOPICS

Cost of Long-Span Reinforced Concrete Arch Roofs—Charles S. Whitney, Ammann and Whitney, N. Y. C.

SYNOPSIS—The general principles and design methods used in long-span arch roofs are discussed. The cost is affected by the location of the shell with respect to the arch ribs. The author describes five important jobs on long-span shell construction and compares the cost of concrete structures with timber and steel.

One or two more papers scheduled for this session, have not materialized at the time this issue goes to press.

WEDNESDAY, FEBRUARY 22

9 a.m.—Ballroom—Research—an open session of Committee 115, Research—S. J. Chamberlin, Chairman; George W. Washa, Secretary.

A group of short papers on research techniques and preliminary results. Papers and discussions not for publication or press release.

2 p.m. — Ballroom — Panel discussion—based on questions submitted by ACI Members headed by C. H. Scholer of Kansas State College.

As this JOURNAL goes to press, panel members have not been named. With Professor Scholer presiding, discussion of top interest is assured with every reason to believe that differences of opinion will make it lively and worthwhile.

Honor Roll

February 1, 1949, to November 30, 1949

One year ago the Morgans held the two top spots on the Honor Roll with a total of 63 credits. Today they are again in the same position but they have already rung up credit for 82½ member applicants.

Will more of our members follow the Morgan pattern? ACI doesn't believe in membership "drives" or "weeks." We do appreciate the efforts of present members to bring ACI membership advantages to the attention of nonmembers who would be benefitted by what ACI is doing and be a benefit to ACI. It is an important part of member responsibility in an organization founded on the idea of a mutual effort. *More* members should increase our effectiveness, enabling us to do more with the force of members.

Don't forget the current Honor Roll ends January 31, 1950.

Newlin D. Morgan (Ill.)	55½
Newlin D. Morgan, Jr. (Wyo.)	27
F. N. Menefee (Mich.)	11½
Jose Antonio Vila (Cuba)	10½
R. H. Sherlock (Mich.)	9
H. J. Gilkey (Ia.)	6½
James A. McCarthy (Ind.)	6
Jaime Alberto Mitrani (Cuba)	6
William T. Neelands (Ga.)	4½
Howard Simpson (Mass.)	4½
Raymond E. Davis (Calif.)	4
F. J. Ochoa U. (Guatemala)	4
J. Vicente Orozco (Mexico)	4
Roberto Barillas F. (Guatemala)	3½
John J. Hogan (N. Y.)	3½
Ralph W. Kluge (Ind.)	3½
Frank E. Richart (Ill.)	3½
Thomas E. Stanton (Calif.)	3½
H. F. Gonnerman (Ill.)	3
Henry E. Griset (N. C.)	3
H. D. Jolley (Ill.)	3
Barton Jones (Puerto Rico)	3
J. W. Kelly (Calif.)	3
Frederic F. Mavis (Pa.)	3
Leon Venegas (La.)	3
Stanton Walker (D. C.)	3
Harold S. Carter (Utah)	2½
R. A. Caughey (Ia.)	2½
Roger H. Corbetta (N. Y.)	2½
Elmo C. Higginson (Colo.)	2½
R. R. Kaufman (Ohio)	2½
A. J. McElrath (Tenn.)	2½
Paul W. Abeles (England)	2
A. Amirikian (Md.)	2
J. F. Baxter (N. Y.)	2
Sterling Lowe Bugg (Fla.)	2
W. S. Cottingham (Wis.)	2
Jacob J. Creskoff (Pa.)	2
John G. Dempsey (N. Y.)	2
Ambrosio R. Flores (Calif.)	2
Zorislav Franjetic (Argentina)	2
Robert J. Hansen (Mass.)	2
J. E. Jellick (Calif.)	2
V. P. Jensen (Calif.)	2
Emory E. Johnson (S. D.)	2
Henry L. Kennedy (Mass.)	2
Karl W. Lemcke (N. Y.)	2
Sam McCluer (Ga.)	2
H. J. McGillivray (Fla.)	2
William S. Moore (Ind.)	2
George G. Nichols (Ore.)	2
William D. Nowlin (Va.)	2
Luis Paredes-Manrique (Colombia)	2
Charles Peterson (Calif.)	2
Luis A. Pietri-Lavie (Venezuela)	2
Dan H. Pletta (Va.)	2
Edward W. Scripture Jr. (Ohio)	2
Thomas C. Shedd (Ill.)	2
Frederick E. Springate (Calif.)	2
Manabu Tanaka (Hawaii)	2
G. E. Troxell (Calif.)	2
George Winter (N. Y.)	2
S. J. Chamberlin (Ia.)	1½
Leo H. Corning (Ill.)	1½
H. C. Delzell (Ill.)	1½
A. H. Douglas (Canada)	1½
Adly W. Gindy (Egypt)	1½
Adli Milik Hanna (Egypt)	1½
Edgar R. Kendall (Nebr.)	1½
Robert L. Mauchel (N. Y.)	1½
Harmon S. Meissner (Colo.)	1½
Carl A. Menzel (Ill.)	1½
Roy T. Sessums (La.)	1½
Bertrand H. Wait (N. Y.)	1½
Calvin T. Watts (La.)	1½
Frank W. Wheeler (Va.)	1½
A. J. Abrams	1
Jerome O. Ackerman	1
William F. Anderson	1
Harold Austin	1
Samuel Baker	1
J. A. Bakker	1
Charles F. Ball	1
John H. Bateman	1
Donald C. Beam	1
Oscar Benedetti	1
H. J. Bezette	1
Robert F. Blanks	1
Hans H. Bleich	1
Ernest M. Bowles	1

R. W. Brandley 1
Carlos Mora Bravo 1
Milton Brumer 1
R. Y. Bush 1
Adraino Carbone 1
D. R. Cervin 1
R. V. Chate 1
H. Clare, Jr. 1
Miles N. Clair 1
E. F. J. Clark 1
H. W. Comack 1
R. E. Copeland 1
Alirio Cornejo 1
Hendrik Cornelissen 1
G. H. C. Crampton 1
Ray A. Crysler 1
Jose A. Cuevas 1
M. H. Cutler 1
E. Davis 1
Harmer E. Davis 1
W. Dewoody Dickinson, Jr. 1
Peter J. Doanides 1
A. T. Docal 1
Ernest A. Dockstader 1
W. H. Droege 1
C. Martin Duke 1
Belmon U. Duvall 1
John J. Dwyer 1
Harlan H. Edwards 1
J. M. Faircloth 1
Ewell W. Finley 1
F. L. Fitzpatrick 1
George W. Ford 1
Carl Fox 1
Robert W. Freeman 1
Carmelo Galindez 1
Gerald K. Gillan 1
Castor Segunda Goa 1
Ali N. Gokyigit 1
E. Gonzalez-Rubio 1
Armour T. Granger 1
Samuel J. Green 1
J. E. Guest 1
K. Hajnal-Konyi 1
Robert C. Hanckel 1
George E. Hatch 1
Bernard C. Herring 1
M. D. Hicklin 1
Percy Howat 1
Franklin D. Howe 1
Myron A. Howe 1
J. W. Hubler 1
Alberto Dovali Jaime 1
J. P. Jain 1
V. P. Jensen 1
R. C. Johnson 1
Oliver G. Julian 1
Milton Kellert 1
Frank Kerekes 1
K. V. Kini 1
Reginald A. Kirkpatrick 1
Max Krueger 1
Blas A. Lamberti 1
George E. Large 1
William Lerch 1
Leslie D. Long 1
Winthrop E. Luke 1
E. Lyons 1
Charles Macklin 1
Ian MacTaggart 1
Leon C. Magbanua 1
Glenway Maxon 1
George C. McLeod 1
F. R. McMillan 1
M. J. McMillan 1
Jesus Mesa 1
Rafael Garcia Montes 1
Robert B. B. Moorman 1
Robert Morris 1
R. F. Moss 1
LeRoy T. Oehler 1
E. L. Oliver 1
Czeslaw Ozieblowski 1
Orin G. Patch 1
Jose F. Perez 1
Anthony F. Pessolano 1
A. Edward Piller 1
James M. Polatty 1
W. S. Powell 1
Emil Praeger 1
Gayle B. Price 1
Evan L. Richard 1
Raul Miguel Rivero 1
J. E. Robinson 1
Paul Rogers 1
Georges Ronai 1
Albert Rootberg 1
John A. Ruhling 1
Carl E. Sandstedt 1
Emil Schmid 1
Wilfrid Schnarr 1
Philo H. Schultz 1
F. Selmer 1
Saul Shaw 1
Isaac S. Shina 1
Ranbir Singh Shrikant 1
Richard Skalak 1
J. Morgan Smith 1
Myron M. Smith 1
William Furber Smith 1
Luis G. Solano A. 1
Leroy A. Staples 1
Alois Starkmann 1
Martin Steljes 1
Jiri Stork 1
W. B. Summer 1
E. O. Sweetser 1
Maurice Taylor 1
T. W. Thomas 1
I. L. Tyler 1
Oscar J. Vago 1
Joseph J. Waddell 1
William R. Waugh 1
John M. Wells 1
J. G. Welsh 1
Donald K. Woodin 1

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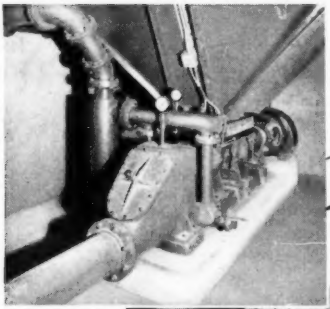

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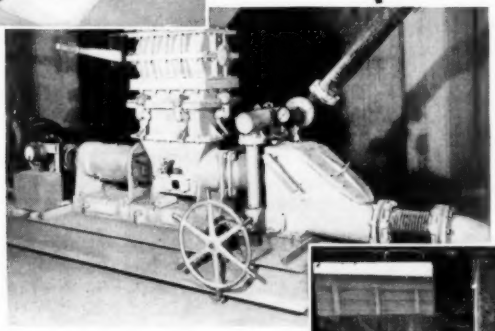

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P-112

Who's Who This Month

Robert C. Hanckel

Robert C. Hanckel, Consulting Engineer, Boston, Mass., presented "Precast Units for Short-Span Bridges," p. 317, at the ACI New England Regional Meeting, held last November in Boston.

Mr. Hanckel as a consulting engineer specializes in structural designs for all types of buildings. During World War II he was associated with the Stone and Webster engineering firm and later with the Thompson and Lichtner Co. as principal design engineer in charge of all structural design, which included the design of the unique apron building for the East Boston Airport. It was during this period that Mr. Hanckel became conversant with precast concrete construction.

Receiving his B. S. in Civil Engineering in 1917 from the Worcester Polytechnic Institute, he served with the United States Army during World War I and later spent a short time in Costa Rica. Returning to the United States, he was employed in the construction field and also as a sales engineer.

Mr. Hanckel was employed by the New York State Department of Architecture from 1929 to 1941 except for 2½ years spent with the Public Buildings Branch of the United States Treasury in Washington.

As a structural engineer with New York State and the United States, Mr. Hanckel was in responsible charge of design for all types of buildings including hospitals, schools, armories, prisons and many other structures which used large quantities of reinforced concrete for continuous frameworks. (It was during this period he met Walter H. Wheeler and has since become an ardent disciple of Mr. Wheeler's system of flat slab design.)

Mr. Hanckel is a member of ACI, A.S.C.E. and the Massachusetts Society of Professional Engineers, and is a licensed professional engineer in all New England States and New York.

Kenneth B. Woods

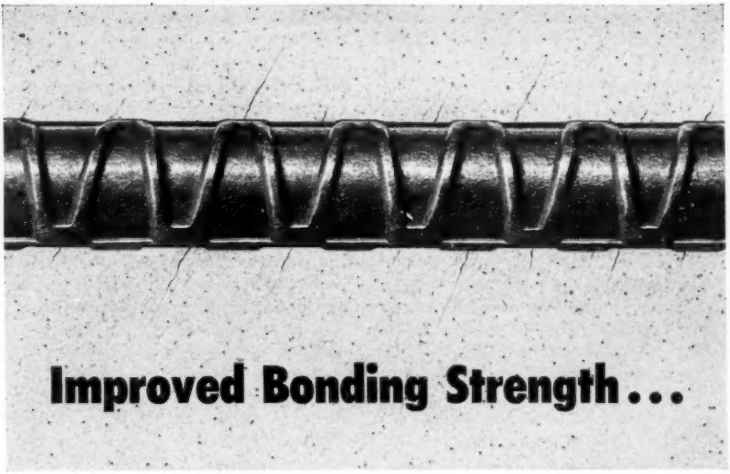
Kenneth B. Woods, Professor of Highway Engineering and Associate Director of the Joint Highway Research Project, Purdue University, Lafayette, Ind., is the author of "Influence of Subgrades and Bases on Design of Rigid Pavements," p. 329, scheduled to be presented at the ACI 46th Annual Convention in February.

An ACI Member, Prof. Woods was coauthor with H. S. Sweet of "Evaluation of Aggregate Performance in Pavement Concrete," June 1948, JOURNAL.

In 1945 he received the Annual Highway Research Board Award for a paper of outstanding merit in the field of highway materials, entitled "Pavement Blow-ups Correlated with Source of Coarse Aggregate."

He joined the staff of the School of Civil Engineering and Engineering Mechanics, Purdue University, in 1939 and his service at Purdue has been continuous since that date.

Prof. Woods' research endeavors have been in the field of highway and airport engineering. As Associate Director of the Joint Highway Research Project, a cooperative project between the Indiana State Highway Commission and Purdue University, which is currently operating on an annual budget of $100,000, he is in immediate charge of a wide range of highway research projects including studies in concrete technology, soil studies, pavement performance investigations and traffic studies. The contributions of this organization in the development of techniques for the interpretation of soil types and land forms from airphotos have gained national recognition. Likewise, the work of this organization in the field of pave-



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ment pumping and the work on the durability characteristics of Indiana aggregates has been outstanding.

The research organization under his direction has also participated extensively in cooperative research endeavors with the Civil Aeronautics Authority and with the Corps of Engineers.

Prof. Woods obtained his academic training at Ohio State University, receiving the degree of B. C. E. in 1932 and the professional degree, C. E., in 1937.

He served for three years on the instructional staff of the Department of Civil Engineering at Ohio State. From 1933 to 1939 he was employed as soils engineer by the Ohio Department of Highways.

Professor Woods is also a member of A.S.C.E., A.S.T.M., American Road Builders' Assn., Highway Research Board and the Indiana Society of Professional Engineers. He has served on many of the technical committees of these societies and contributed extensively to their technical journals.

Lewis H. Tuthill

Lewis H. Tuthill, U. S. Bureau of Reclamation, Denver, Colo., author of "Inspection of Mass and Related Concrete Construction," p. 349, is well-known to JOURNAL readers.

Awarded the ACI Construction Practice Award for his 1945 paper "Concrete Operations in the Concrete Ship Program," Mr. Tuthill has been an active ACI Member since 1926. He has had seven papers published in the JOURNAL as well as serving as chairman of Committee 614, whose report on "Recommended Practice for Measuring, Mixing and Placing Concrete," became an ACI Standard in 1942.

He was a member of the Advisory Committee 1944-46 and the ACI Honor Roll records attest that he has been an active worker for new ACI Members.

Since graduation from Oregon State College in 1920, Mr. Tuthill has been continuously engaged in design and construction work on irrigation, water supply projects and the building of dams, with the technique and control of concrete as his primary and absorbing interest.

His first employment was with the Grants Pass Irrigation District in Oregon; next with the Merced Irrigation District in California, where he was associated with the control of concrete on Exchequer Dam; and, following this, the same detail on Malones Dam for the Oakdale and South San Joaquin Irrigation District. After a year of private consulting practice, concrete control work on Pardie and Hogan Dams next occupied his attention.

During seven years with the Metropolitan Water District of Southern California, Mr. Tuthill prepared specifications for 4½ million yards of concrete work and materials, passed upon the acceptance of cement and materials for the aqueduct from the Colorado River to the Los Angeles area, and prepared a manual of inspection.

Since 1939 he has been a member of the staff of the Bureau of Reclamation, still pursuing the profession of concrete engineering and assisting in editing the Bureau's Concrete Manual. During a leave of absence from the Bureau, he spent several years during the war supervising concrete quality on the U. S. Maritime Commission's concrete ship construction program.

J. M. Wells

"Experimental Grouting Investigation for Chief Joseph Dam," p. 361, is the first contribution to JOURNAL pages by J. M. Wells, Chief of the Materials Control Section, Corps of Engineers, McNary Dam, Ore.

Mr. Wells joined the Corps of Engineers in 1942 as a civil engineer. He headed the Special Investigations Section in the North Pacific Division with responsibility for the location of airfields and other military installations in the Pacific Northwest.

Following the war, Mr. Wells was in charge of the Seattle District concrete laboratory and conducted construction

materials surveys in connection with the Columbia River "308" Report. Upon initiation of construction at McNary Dam on the lower Columbia River, he was transferred to this project in his present capacity.

Mr. Wells has been connected with concrete and materials investigations since 1928. He spent seven years as a testing engineer with one of the leading commercial testing laboratories on the Pacific Coast before joining the concrete control section of the U. S. Bureau of Reclamation. The next seven years were spent in the capacities of inspector, assistant engineer and associate engineer in concrete control which included aggregate studies on such projects as Grand Coulee Dam, Shasta Dam and Friant Dam.

Raymond E. Davis

Another paper presented at the ACI New England Regional Meeting in Boston was "Use of Pozzolans in Concrete," p. 377, by Raymond E. Davis, Director, Engineering Materials Laboratory, University of California, Berkeley, Calif.

Prof. Davis, past president of the Institute, author or co-author of more than a dozen ACI papers, twice a Wason medalist (1931, 1934) and winner of the Construction Practice Award (1948) scarcely needs a long introduction to regular JOURNAL readers.

He and Harmer E. Davis were presented the Wason Medal for the most meritorious paper for their 1931 paper "Flow of Concrete Under the Action of Sustained Loads." The paper, "Cement Investigations for Boulder Dam with the Results up to the Age of One Year," by Prof. Davis, R. W. Carlson, G. E. Troxell and J. W. Kelly, won the same award in 1934. The 1948 Construction Practice Award went to Prof. Davis, E. Clinton Jansen and William T. Neelands for "Restoration of Barker Dam."

A member since 1926, he was president of the Institute in 1942; vice-president, 1940-41; Board of Direction member for 18 years, 1930-48; Advisory Committee, 1929-44; Program Committee, 1932-40; and Publications Committee, 1939-46. Prof. Davis has also been active on the technical committees of the Institute, and he was winner of the membership-getting race in the 1947 Honor Roll.

Graduating from the University of Maine in 1911, Prof. Davis taught for seven years at the University of Illinois, receiving his M. S. degree there in 1916. After a year as associate professor of civil engineering at the University of Nebraska he joined the staff of the University of California in 1920 where he is now professor of civil engineering and director of the Engineering Materials Laboratory.

He is also affiliated with A.S.C.E., A.S.T.M., Sigma Xi, Tau Beta Pi, Chi Epsilon and Delta Sigma Chi. He is a fellow of the American Assn. for the Advancement of Science.

ACI visitor

Musaddi Lal, Engineer, Public Health Department, Government of United Provinces, India, was a recent visitor at Institute headquarters. Mr. Lal is on a tour of the United States to investigate the latest developments in reinforced concrete structures, housing and sanitary engineering.

New Members

New York was top membership state for the month of November with six new member applicants. There was a three-way tie for second place honors among California, Illinois and Guatemala, each providing five applicants.

40 percent of November's members are outside of the United States.

Our membership total became 4853 on December 1.

California

Alborg, Willis J., (Indiv.) 808 N. Keystone St., Burbank, Calif.
Alvy, Reuben R., (Indiv.) 5702½ Clemson St., Los Angeles 16, Calif.
Layne, Henry M., (Indiv.) 2500 W. 6th St., Los Angeles 5, Calif.
Pollman, Albert P., (Indiv.) U. S. Atomic Energy Comm., P. O. Box 559, Berkeley, Calif.
Raju, P. S. Ramakrishna, (St.) 1731 Allston Way, Berkeley 3, Calif.

Florida

Hart, J. L., (Indiv.) Hart Concrete Products Co., P. O. Box 1260, Tampa, Fla.

Georgia

Jackson, Melvin W., (Indiv.) Georgia Institute of Technology, Atlanta, Ga.
Patchen, Josef C., (Indiv.) Patchen & Zimmerman, Engrs., Daniel Field Airport, Augusta, Ga.

Illinois

Barnett, E. Frances, (Jr.) Dept. of Filtration Design, Room 1100, 220 S. State St., Chicago 4, Ill.
Gardner, Junius R., (Indiv.) 253 S. Park St., Decatur, Ill.
Gregg, Richard S., (Indiv.) 300 1st National Bank Bldg., Peoria, Ill.
Steinour, Harold H., (Indiv.) 1829 Elm Ave., Northbrook, Ill.
Zancaner, Arnaldo, (St.) 209 E. Healey St., Champaign, Ill.

Indiana

Eskenazi, Nisim Israel, (St.) 333 Stadium Ave., W. Lafayette, Ind.
Noronha, Leslie, (St.) 503 Morrissey Hall, University of Notre Dame, Notre Dame, Ind.

Iowa

Wells, J. Perry, (Indiv.) 1900 Delaware, Des Moines 17, Ia.

Massachusetts

Adam, James Jr., (Indiv.) 34 Garrison Rd., Wellesley 81, Mass.
Bilolikar, Ganpat Rao, (St.) The Graduate House, MIT, Cambridge 39, Mass.
Dell'Isola, Alphonse John, (St.) 165 Vine St., Everett 49, Mass.
Schwarz, Alfred, (Indiv.) 11 Allison St., Boston 14, Mass.

Minnesota

McComb, Fred R., (Indiv.) District Engineer, Portland Cement Association, Northwestern Bank Bldg., Minneapolis 2, Minn.

New York

Alpern, Milton, (Jr.) The Cooper Union, Dept. of Civil Engineering, Cooper Square, New York, N. Y.
Emerson, H. H., (Indiv.) 60 Myrtle St., LeRoy, N. Y.
Itak, John M., (Indiv.) 816 Seneca Ave., Ridgewood 27, N. Y.
Otto, Henry S., (Indiv.) P. O. Box 523, Scarsdale, N. Y.
Prehy Co., Inc., (Corp.) 420 Lexington Ave., New York 17, N. Y. (Peter B. Bradley)
Reeve, Edmund W., (Indiv.) c/o Alphons Custodis Chimney Construction Co., 25 Broadway, New York 4, N. Y.

North Carolina

Knott, William Archibald, (St.) 1617½ Oberlin Rd., Raleigh, N. C.

Ohio

McMinn, Fred Francis, (Indiv.) Room 328, City Hall, Cincinnati 2, Ohio

Oregon

McCarthy, R. J., (Indiv.) 550 N. Summer, Apt. 109, Salem, Ore.

Pennsylvania

Hoopes, Charles Herbert, (Jr.) 413 N. 6th St., Allentown, Pa.
Kerr, George C., (Indiv.) 215 Bauman Ave., Pittsburgh 27, Pa.
Ritter, Clarence H., (Indiv.) Arenel Farms, Shoemakersville, Pa.

Texas

Sutton, H. G. Jr., (Jr.) P. O. Box 778, Colmesneil, Texas

Virginia

Rhodes, James A., (Indiv.) 510 Birmingham Ave., Norfolk 5, Va.

Washington

Brunner, Fred H., (Indiv.) Corps of Engrs., Seattle District, c/o Foundation & Materials Section, 4735 E. Marginal Way, Seattle 4, Wash.

Wisconsin

Strand, J. A., (Indiv.) Consulting Engineer, 1 W. Main St., Madison 3, Wis.

Australia

Howard, Keith Alister, (Indiv.) Bogong, Via Wodonga, Victoria, Australia

Martin, Birk C., c/o C. H. Martin Ltd., 82 Charles St., Unley, S. Australia

Bahamas

Lothian, B. J., (Indiv.) P. O. Box 1111, Peek Bldg., Nassau N. P., Bahamas

Canada

Lindsay, Guy A., (Indiv.) Engineer-in-Charge, General Engineering Branch, Dept. of Transport, Ottawa, Ont., Canada

Moore, Stanley, (Indiv.) Box 340, Tunnel Development, c/o Hydro-Electric Power Comm., Thessalon, Ont., Canada

Rogers, James Donald, (St.) 21 Vernon St., Halifax, N. S., Canada

Cuba

del Campo y Delfin, Jose Fernandez, (Indiv.) Mayia Rodriguez 170, Santos Suarez, Havana, Cuba

Poveda, Arquimedes, (Indiv.) Edificio Fren-Mar, Calle 3a Esq. 2, Vedado, Havana, Cuba

England

Cancea, Charles L., (Indiv.) 119 Victoria St., Westminster, S. W. 1, England

Clifford, W. J., (Indiv.) The Pyrene Co. Ltd., Great West Road, Brentford, Middlesex, England

Lawson, Stanley P., (Indiv.) 197, Burton Rd., W. Didsbury, Manchester 20, U. K., England

Tsai, Y. P., (Indiv.) c/o Matthews & Mumby Ltd., 129 Stockport Rd., Manchester 12, England

Germany

Graf, Otto, (Indiv.) Stuttgart, Spittlerstrasse 30, Germany

Guatemala

Barillas F., Gonzalo, (St.) 17 C. O. 9, Guatemala City, Gautemala

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Positions and Projects — ACI Members

Manning retires; Jelley heads Budocks

Capt. Joseph F. Jelley, Jr., Civil Engineer Corps, U. S. N., has been appointed as Chief of the Navy's Bureau of Yards and Docks and Chief of Civil Engineers, succeeding Rear Admiral John J. Manning, CEC, U. S. N.

Rear Admiral Manning retired from active duty and accepted the position of Executive Vice-President with John McShain, Inc., Philadelphia, Pa. A member of the Civil Engineer Corps since 1917, he held numerous important assignments, having been director of the Atlantic division of the bureau from 1942 to 1945 and director of the Eastern Pacific division from the latter date to 1946.

Captain Jelley, who has been serving as Deputy Chief of Bureau since 1946, announced that Captain Archibald D. Hunter, CEC, U. S. N., would fill the Deputy post. Captain Hunter has been serving on the staff of the Chief of Naval Air Training, Pensacola, Fla.

Captain Jelley played a key role as Deputy Chief of Bureau in directing the post-war roll-up of the Navy's wartime shore establishments and the construction of permanent peacetime structures, principally housing, hospitals and research facilities. Captain Hunter is best known for his work at Pearl Harbor during the early months of the war.

The three men are ACI Members.

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Amirikian receives welding medal

A. Amirikian, head engineer, Bureau of Yards and Docks, Navy Dept., Washington, D. C., was awarded the 1949 Lincoln Gold Medal by the American Welding Society for his paper "Future Developments in Welded Steel Buildings," published in the *Welding Journal*.

Mr. Amirikian, an active ACI Member, has had several papers published in the ACI JOURNAL, including "Precast Concrete Structures," "Some Problems in Structural Framing of Precast Concrete Houses" and "Precast Concrete Storehouses." He also is chairman of Committee 324, Precast Reinforced Concrete Structures.

Orin G. Patch awarded medal

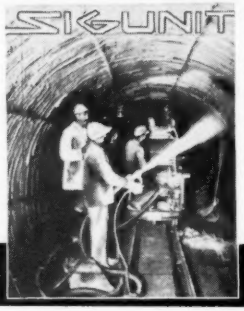
Orin G. Patch, retired chief concrete engineer of the U. S. Bureau of Reclamation's Columbia River district at Coulee Dam, Wash., has been awarded the Department of the Interior Gold Medal and Distinguished Service Certificate. Among his accomplishments were: the development of a method for the accelerated curing of concrete cylinders; and development of a meter to determine percentage of air entrained in concrete.

Stone and Webster safety award

The Stone and Webster Engineering Corp., Boston, Mass., established the William N. Patten Trophy recently in recognition of Mr. Patten's leadership in accident prevention work during his 50 years of association with the firm.

The trophy will be kept permanently at the offices of Stone and Webster, annual awards being made and inscribed on it for the Stone and Webster construction project having an outstanding safety record for the year.

Mr. Patten, at present a director of the firm, intensified safety activities on the construction projects with which the



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firm was associated with a year by year reduction in the frequency and severity of disability accidents.

Cortelyou retires

Spencer V. Cortelyou, assistant state highway engineer for the California Division of Highways in charge of District VII, including the metropolitan area of Los Angeles, has retired after 38 years of service. Prior to entering the Division of Highways as principal assistant division engineer of District VII, Mr. Cortelyou had been chief of the survey party and the Los Angeles County Road Dept. of Public Works, and earlier was with the Insular Dept. of Public Works of the Philippines. He played a vital part in the planning and construction of many major arteries of travel in California.

Huron announces building program

The Huron Portland Cement Co., ACI Contributing Member, Detroit and Alpena, Mich., announced a new building program to enlarge existing facilities. The first part of the program, started in 1946 and completed in 1949, covered the installation of four new kilns and necessary equipment to make the Alpena installation the world's largest single cement producing mill. Another self-unloading bulk cement carrier was also added to the lake fleet of the Huron Transportation Co.

Emory M. Ford, president of the company, stated that two new kilns will be added at Alpena in 1950. A new plant will be built at Superior, Wis., which will process cement clinkers from the Alpena plant, as well as the completion of a modern distributing plant at Green Bay, Wis., and additional storage silos at Muskegon, Mich. The program also calls for additional equipment at the mill and distributing plants.

Concrete aggregate short course completed

The Fourth Annual Short Course on Aggregates and Concrete, sponsored by the University of Maryland, the National Ready-Mixed Concrete Assn. and the National Sand and Gravel Assn., was held at the University of Maryland with an enrollment of 102. The first course, held in 1946, had an attendance of 31.

Sessions consisted of lectures, demonstrations, calculation periods and class discussions on the technology of aggregates and concrete.

Bridge competition features prestressing

A competition for designs of prestressed and reinforced concrete bridges is being sponsored by the Cement and Concrete Assn., London, England. The competition is open only to engineers and architects in Great Britain and Northern Ireland.

Competitors must submit one design for each of three bridges: (1) a highway bridge having a width of 42 ft and span of 73 ft without central support; (2) a highway bridge of two or more spans with a width of 42 ft; and (3) a footbridge 6 ft wide.

The designs may be for any type of bridge constructed in plain, reinforced or prestressed concrete or a combination of these materials and may be completely or partially precast.

California highway conference

The Second California Institute on Street and Highway Problems will be held at the University of California at Los Angeles from Jan. 30 to Feb. 1. This is the second annual statewide conference on streets, roads and highways. It is sponsored by the Institute of Transportation and Traffic Engineering in cooperation with University of California Extension. The first conference, held last year in Berkeley, had 350 in attendance. Heading this year's planning committee is R. M. Gillis, Assistant State Highway Engineer, Sacramento, Calif. The program for the 1950 meeting includes papers by prominent road men from throughout the nation, as well as leading Californians.

Vacuum concrete investigation

In connection with the Civil Works Investigations Program of the Office, Chief of Engineers, the Waterways Experiment Station has been making tests to determine proper procedures and techniques for applying vacuum treatment to both formed and unformed surfaces of mass concrete. The vacuum treatment of concrete entails the use of equipment and methods patented by Vacuum Concrete, Inc., Philadelphia, Pa. This company has licensed the Corps of Engineers to use vacuum treatment on civil works construction.

Seven horizontal slabs and a test block were built in which many of the features of a mass concrete dam are reproduced. The test block contains 133.3 cu yd of concrete and is analogous to a portion of a dam. The concrete bulkhead wall form was selected to permit simulation of "job" sealing conditions both against steel form bulkheads for "leading" monoliths and against hardened concrete for "following" monoliths. The test block has a vertical "upstream" face and an inclined "downstream" face and was constructed in three 5-ft lifts. Cantilever steel forms were used in the construction of the test block and were furnished by the Blaw-Knox Co., Pittsburg, Pa.

Detailed observations were made of all operations. Samples were taken for tests both on the plastic and hardened concrete. The test slabs and block were built and tests are being conducted under the direction of Lt. Col. R. D. King, Director of the Waterways Experiment Station, and Herbert K. Cook, Chief of the Concrete Research Division.

Hirschthal retires

After 42 years service with the Delaware, Lackawanna and Western Railroad, M. Hirschthal, New York, retired to enter consulting engineering practice. He had held the position of concrete engineer since 1920. Mr. Hirschthal was a member of the ACI Board of Direction, 1940-41, and was the author of the JOURNAL contribution "Maintenance and Repair of Concrete Structures in Railroad Construction," Jan.-Feb. 1937 issue.

He received his B.S. at the College of the City of New York in 1899 and C.E. degree from Columbia University in 1902. Mr. Hirschthal was associated with Ernest C. Ransome, Roebling Construction Co. and Henry Steers, Inc. before joining the D. L. & W. R. R. as a designer. He was appointed assistant engineer in 1915 and concrete engineer in 1920. He has been an ACI Member since 1934.

Experimental sidewalk

The Civil Engineering Department at Northeastern University, Boston, Mass., is hoping for an old-fashioned New England winter to test their experimental

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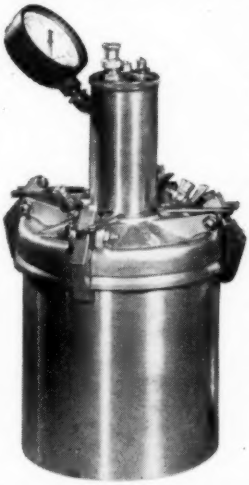

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sidewalk. A "healthy" freezing and thawing cycle is needed to test 117 blocks of different concretes that form the sidewalk.

The experiment is being made to determine the optimum mix of concrete for paving purposes and to test the efficacy of various curing methods and of air-entraining agents.

Other tests on the concrete blocks are carried out in the department's temperature and humidity controlled laboratory where samples of varying size are placed in a 10 percent solution of salt and frozen for 24 hours and thawed for 24 hours. (600 lb of concrete are frozen every night.) Sonic modulus and weight-loss measurements are made throughout the freezing and thawing cycle.

Thomson joins Material Service Corp.

Harry F. Thomson, ACI vice-president, has been retained by the Material Service Corp. of Chicago to be in over-all charge of their extensive ready-mixed concrete service recently inaugurated in the Chicago area.

Mr. Thomson was formerly president of the General Material Co. in St. Louis, one of the pioneer companies in the ready-mixed field.

He has been active in ACI affairs since 1928, his latest paper being "Specifications Should Be Realistic," in the November JOURNAL. Mr. Thomson was elected to the ACI Board of Direction in 1939 and served until 1940, then again from 1942-48, and was elected vice-president in 1949.

Mr. Thomson served as president of the National Ready Mixed Concrete Assn., and is also a former national director of A.S.C.E.

Are YOU "MEMBER CONSCIOUS" ? ? ?

A. W. K. Billings

A. W. K. Billings, retired president of the Brazilian Traction, Light and Power Co., Ltd., Toronto, Canada, and Rio de Janeiro, Brazil, died recently. Mr. Billings, an ACI Member since 1913, became affiliated with the Brazilian Traction, Light and Power Co. in 1924, serving successively as vice-president and president, and retiring in 1948.

He was associated with the Canadian and General Finance Co., Ltd., Toronto, from 1921 to 1924. Prior to that he was with the Ebro Irrigation and Power Co., Ltd., Barcelona, Spain.

New Members

Continued from p. 21

Ponce Navarro, Guillermo, (St.) 8a Avenida sur No. 69, Guatemala City, Guatemala

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Zepeda Aldana, Roberto, (St.) 7a Calle Oriente No. 45, Guatemala City, Guatemala

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Omura, Shizuo, (Indiv.) c/o East Maui Irrigation Co., Ltd., P. O. Box 196, Paia, Maui, T. H.

India

Acharya, V. H., (Indiv.) Victoria Jubilee Technical Institute, Matunga, Bombay 19, India

Mexico

Lazcano, Enrique Rios, (Indiv.) Prado Norte No. 662, Lomas de Chapultepec, Mexico D. F., Mexico

New Zealand

B & B Concrete Co., Ltd., (Corp.) P. O. Box 72, Newmarket, Auckland, N. Z. (G. M. K. Hagen, Managing Director)

Philippines

Sun, Denis A., (Indiv.) 25 P. Arroyo St., Iloilo City, Philippines

S. Rhodesia

Stevens, Kenneth Gordon, (Indiv.) P. O. Box 1669, Salisbury, S. Rhodesia

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Continued from p. 14

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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 21 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES (ACI 616-49) 46-1

Price 50 cents (in special covers).

REPORT of COMMITTEE 616—Sept. 1949, pp. 1-16 (V. 46)

Supersedes 38-30 and 45-18.

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

AN ULTRASONIC METHOD OF STUDYING DETERIORATION AND CRACKING IN CONCRETE STRUCTURES 46-2

Price 35 cents.

J. R. LESLIE and W. J. CHEESMAN—Sept. 1949, pp. 17-36 (V. 46)

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

MANUFACTURE OF REINFORCED FOAM CONCRETE ROOF SLABS ... 46-3

Price 35 cents.

I. T. KOUDRIASHOFF—Sept. 1949, pp. 37-48 (V. 46)

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

SUGGESTIONS ON CONCRETE FLOOR CONSTRUCTION 46-4

Price 35 cents.

ERNST GRUENWALD—Sept. 1949, pp. 49-56 (V. 46)

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

USE OF AIR-ENTRAINING CONCRETE IN CANAL LINING 46-5

Price 35 cents.

JOSEPH J. WADDELL—Sept. 1949, pp. 57-64 (V. 46)

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

THE USE OF PORTLAND-POZZOLAN CEMENT BY THE BUREAU OF RECLAMATION .. 46-6

Price 35 cents.

ROBERT F. BLANKS—Oct. 1949, pp. 89-108 (V. 46)

The Bureau of Reclamation has made extensive studies of pozzolanic materials, and portland-pozzolan cements are now being used in many of the major structures built by the Bureau. The properties of portland-pozzolan cement that are used advantageously in the production of mass concrete are described.

RESISTANCE OF CONCRETE AND PROTECTIVE COATINGS TO FORCES OF CAVITATION 46-7

Price 35 cents.

WALTER H. PRICE and GEORGE B. WALLACE—Oct. 1949, pp. 109-120 (V. 46)

A machine used for producing cavitation erosion in the laboratory is described and the results of tests made to investigate the effect of mix proportions, surface treatment, and protective coatings on the resistance of concrete to cavitation are discussed. Through proper use of these, the resistance of concrete surfaces to cavitation erosion may be extended three or four times, but even the best concrete will not resist the forces of cavitation for a prolonged period. Heavy rubber coatings bonded well to the surface of the concrete have proved effective.

VACUUM PROCESSES APPLIED TO PRECAST CONCRETE HOUSES ... 46-8

Price 35 cents.

K. P. BILLNER and BERT M. THORUD—Oct. 1949, pp. 121-128 (V. 46)

The use of vacuum processes in precast concrete construction simplifies the building of fire-resistant, durable structures designed to permit the maximum use of like units. Vacuum processes have thus far been used in one- and two-story houses and one-story industrial or warehouse structures, but further developments should make possible similar benefits for multi-story construction.

The specific vacuum methods utilized are: (1) extracting excess water from freshly placed concrete prior to set, thereby increasing early strength and enabling early handling of units, (2) holding forms in place by vacuum and (3) handling and placing finished and hardened concrete units by cast-in-place closures formed and quickly hardened by vacuum processes.

The use of these methods results in high strength monolithic construction having exterior and interior finished surfaces, with insulation incorporated into the construction, if desired.

NEW TYPE OF CONSISTENCY METER TESTED AT ALLATOONA DAM....46-9

Price 35 cents.

JAMES M. POLATTY—Oct. 1949, pp. 129-136 (V. 46)

Various mechanical devices have been developed for indicating consistency of concrete since the early days of visual inspection after discharge from the mixer. A new type meter to measure the consistency of concrete while it is being mixed was tested at Allatoona Dam and the operation and results are described.

EFFECT OF MIXING SEQUENCE ON THE PROPERTIES OF CONCRETE....46-10

Price 35 cents.

F. L. FITZPATRICK and W. SERKIN—Oct. 1949, pp. 137-140 (V. 46)

The order in which the constituents of concrete (aggregates, cement and water) are combined in the mixing operation, has a significant effect upon the properties of the concrete as to workability, strength, density, surface finish and absorption. Tests are reported.

BOND OF CONCRETE REINFORCING BARS....46-11

Price 35 cents.

ARTHUR P. CLARK—Nov. 1949, pp. 161-184 (V. 46)

The tests reported were made to compare the resistance to slip in concrete (bond) of deformed bars when tested in beams and companion pull-out specimens, to secure information on the effects of size of bar, the type of deformations on the bars and the strength of concrete on the bond. The bars were cast in a horizontal position in all test specimens. The variables were depth of concrete under the bar, length of embedment of the bar in the concrete, strength of concrete and diameter of bar. Slip of the bar was measured at the loaded and free ends. Three tests were made with 2 in. of concrete under the bar, with 15 in. of concrete under the bar and with 3-in., 12-in., and 16-in. embedments.

Bond strengths for the beams and the pull-out specimens were affected similarly by changes in the geometry of the bars and the bond test specimens. They were greater when the bars were near the bottom than when they were near the top of the specimens. The highest bond strengths were obtained with bars having deformations conforming to suggested requirements for maximum spacing and minimum height and providing ratios of shearing to bearing areas less than 10, usually less than 6.

PERLITE AGGREGATE: ITS PROPERTIES AND USES....46-12

Price 35 cents.

J. JOHN BROUK—Nov. 1949, pp. 185-192 (V. 46)

Synthetic expanded volcanic rock, better known as perlite is a fairly recent addition to the lightweight aggregate field. Its use in concrete is governed by weight, gradation, mixing procedure, cellular structure, strength of cell walls, insulating properties, cost, etc. Air entrainment appears to be necessary to give a workable, nonsegregating mix. Perlite aggregate blended with sand can be used in high strength structural concrete and concrete products.

THIN WALL CONCRETE SHIP CONSTRUCTION....46-13

Price 35 cents.

FRANCIS R. MAC LEAY—Nov. 1949, pp. 193-204 (V. 46)

The development of construction methods for placing thin concrete walls is described. After several attempts using the standard method of pouring between double forms, a new method was developed which permitted the successful construction of ¾-in. and 1½-in. concrete walls. Tests were also conducted to determine the practicability as well as the strength of Gunite as a medium to unite precast units into a monolithic ship. Later these methods were used successfully in the construction of a concrete landing craft for the United States Navy.

SPECIFICATIONS SHOULD BE REALISTIC....46-14

Price 35 cents.

HARRY F. THOMSON—Nov. 1949, pp. 205-220 (V. 46)

Specifications for concrete in moderate-sized and lesser construction frequently contain provisions which are ambiguous, conflicting in application, or do not fully recognize local materials. Most of these questionable features result from (1) inadequate information regarding the characteristics of concrete, or (2) use of ready-written specifications without revision for conditions or changes in standard requirements.

Among the features discussed are: specifying both method and result; "frozen" specifications; habitual use of "1-2-4"; multiple provisions for quality; strength without naming consistency; recognition of local materials; use of compression tests; time of placing; bin-test of cement. Numerous quotations from specifications are given, and suggestions offered for bringing the provisions in line with actual conditions.

INSPECTION AND TESTING OF MATERIALS....46-15

Price 35 cents.

NICOLAAS T. F. STADTFELD—Dec. 1949, pp. 237-248 (V. 46)

This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns of the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

FLEXURE OF CELLULAR SHELLS...46-16

Price 35 cents.

F. E. WOLOSEWICK—Dec. 1949, pp. 249-256 (V. 46)

After a brief discussion of the uses of cellular shells and methods available for the solution of design problems, a sample problem is set up using the theorem of least work. From charts the moments are determined and the effect of added stiffeners is assessed.

CEMENT PERFORMANCE IN CONCRETE EXPOSED TO SULFATE SOILS....46-17

Price 35 cents.

L. A. DAHL—Dec. 1949, pp. 257-272 (V. 46)

The Long-Time Study of Cement Performance in Concrete deals with the performance of portland cements in concrete under various conditions of exposure in the field. Among these conditions is exposure to sulfate soils, that is, to the so-called "alkali soils." This part of the investigation is reported in Chapter 5, in which the complete data are given. The present paper describes briefly the work reported in Chapter 5 and the conclusions which have been drawn. Those readers who wish to study the results in greater detail are referred to the more complete report.

EARLY STRENGTH OF CONCRETE AS AFFECTED BY STEAM CURING TEMPERATURES.....46-18

Price 35 cents.

JOSEPH J. SHIDELER and WILBUR H. CHAMBERLIN —Dec. 1949, pp. 273-284 (V. 46)

The testing and results obtained on 990 6 x 12-in. concrete cylinders steam cured for various periods and at temperatures ranging from 100 to 200 F are discussed. Strength results are given for ages ranging from 6 hours to 28 days and strengths of companion specimens moist cured at 70 F are given whenever possible. The information presented was obtained for the Bureau of Reclamation for writing specifications covering the steam curing of precast units such as irrigation pipe.

A SHORTCUT FOR DETERMINING REINFORCEMENT IN REINFORCED CONCRETE.....46-19

Price 35 cents.

V. BOGVAD-CHRISTENSEN—Dec. 1949, pp. 285-292 (V. 46)

This paper presents a graph which gives the complete relationship between moments, thrusts, concrete dimensions, reinforcement and resulting stresses for the reinforced concrete members in question.

PRECAST UNITS FOR SHORT-SPAN BRIDGES.....46-20

Price 35 cents.

ROBERT C. HANCKEL—Jan. 1950, pp. 317-328 (V. 46)

Bridge replacement in Lowell, Mass., where minimum traffic interruption was necessary, led to the adoption of precast reinforced concrete units. Available mobile hoisting equipment limited the maximum weight of sections to about 12 tons which permitted small bridges to be precast as complete structures, while for larger bridges, subassemblies were precast and incorporated into the bridge by a cast-in-place deck. The precasting procedures and construction processes are described and illustrated.

INFLUENCE OF SUBGRADES AND BASES ON DESIGN OF RIGID PAVEMENTS.....46-21

Price 35 cents.

KENNETH B. WOODS—Jan. 1950, pp. 329-348 (V. 46)

This paper has been developed from research data, published reports, and experiences gained by observing the performance of both rigid and flexible pavements—particularly as related to subgrade soil textures and the use of base courses. Structural failures of rigid pavements, caused by large concentrations of exceptionally heavy loads, indicate the need for an evaluation of subgrades and bases in determining the most economical design of rigid slabs.

Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations—in regard to position and soils—the use of base courses must be evaluated against the economy of using slabs of increased thickness, more reinforcing steel, or combinations of the two.

It is concluded that it is not entirely feasible, with the present state of knowledge, to standardize rigid pavement design. Rather, the available data indicate that design practices should be developed by regions in which the subgrade soil, availability and type of base course materials, climatic conditions, and traffic volumes and loads are evaluated.

INSPECTION OF MASS AND RELATED CONCRETE CONSTRUCTION.....46-22

Price 35 cents.

LEWIS H. TUTHILL—Jan. 1950, pp. 349-360 (V. 46)

This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual activities of inspection and those associated with concrete control since both have the same objective: a serviceable and presentable structure. Details of requirements and procedure to this end are discussed.

EXPERIMENTAL GROUTING INVESTIGATION FOR CHIEF JOSEPH DAM.....46-23

Price 35 cents.

J. M. WELLS—Jan. 1950, pp. 361-376 (V. 46)

This paper describes the development of a method for the control of seepage into the excavation areas of the right abutment, and through the right abutment, at the Chief Joseph Dam project on the upper Columbia River in the State of Washington.

Laboratory studies are presented on various types of grout mixtures, as well as techniques and procedures for drilling in coarse gravel by jetting methods, procedure for grouting pervious gravels and the efficacy of grouting treatment.

USE OF POZZOLANS IN CONCRETE.....46-24

Price 35 cents.

RAYMOND E. DAVIS—Jan. 1950, pp. 377-384 (V. 46)

After stating the characteristics of pozzolonic materials, the effects of replacing with pozzolans part of the portland cement in concrete are considered briefly. Results of tests with fly ash and superfine diatomite are cited to show the possible use of these materials in the East and Midwest where natural pozzolans of the West are not economically available.

DISCUSSION

Discussion closed January 1, 1950

Sept. Jl. '49

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)—Report of Committee 616

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures—J. R. Leslie and W. J. Cheesman

Manufacture of Reinforced Foam Concrete Roof Slabs—I. T. Koudriashoff

Suggestions on Concrete Floor Construction—Ernst Gruenwald

Use of Air-Entraining Concrete in Canal Lining—Joseph J. Waddell

Discussion closes February 1, 1950

Oct. Jl. '49

The Use of Portland-Pozzolan Cement by the Bureau of Reclamation—Robert F. Blanks
Resistance of Concrete and Protective Coatings to Forces of Cavitation—Walter H. Price and George B. Wallace
Vacuum Processes Applied to Precast Concrete Houses—K. P. Billner and Bert M. Thorud
New Type of Consistency Meter Tested at Allatoona Dam—James M. Polatty
Effect of Mixing Sequence on the Properties of Concrete—F. L. Fitzpatrick and W. Serkin

Discussion closes March 1, 1950

Nov. Jl. '49

Bond of Concrete Reinforcing Bars—Arthur P. Clark
Perlite Aggregate: Its Properties and Uses—J. John Brouk
Thin Wall Concrete Ship Construction—Francis R. Mac Leay
Specifications Should Be Realistic—Harry F. Thomson

Discussion closes April 1, 1950

Dec. Jl. '49

Inspection and Testing Materials—Nicolaas T. F. Stadtfeld
Flexure of Cellular Shells—F. E. Wolosewick
Cement Performance in Concrete Exposed to Sulfate Soils—L. A. Dahl
Early Strength of Concrete as Affected by Steam Curing Temperatures—Joseph J. Shideler and Wilbur H. Chamberlin
A Shortcut for Determining Reinforcement in Reinforced Concrete—V. Bogvad-Christensen

Discussion closes May 1, 1950

Jan. Jl. '50

Precast Units for Short-Span Bridges—Robert C. Hanckel
Influence of Subgrades and Bases on Design of Rigid Pavements—Kenneth B. Woods
Inspection of Mass and Related Concrete Construction—Lewis H. Tuthill
Experimental Grouting Investigation for Chief Joseph Dam—J. M. Wells
Use of Pozzolans in Concrete—Raymond E. Davis

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The long-span concrete arch hangar at Limestone Air Force Base contains several unique features from the standpoint of concrete construction.

Construction of Long-Span Concrete Arch Hangar at Limestone Air Force Base*

By JOHN E. ALLEN†

SYNOPSIS

The reinforced concrete arch hangar at Limestone Air Force Base is noteworthy for its size, being one of the largest of this type yet constructed in this country. The superstructure is composed of a thin reinforced arch shell, shaped like an inverted catenary. Construction procedures are described.

HANGAR DESCRIPTION

Two large reinforced concrete hangars have been completed this past year, at Limestone, Me., and at Rapid City, S. D. They were designed and built under the direction of the Corps of Engineers for the U. S. Air Force. These hangars are of reinforced concrete arch construction and are noteworthy for their size, being the largest of this type yet constructed in this country.

The two hangars differ only in their substructures which were adapted to the foundation conditions at the two sites. The Rapid City structure is supported on piles with tie rod connections between arch abutments while the Limestone hangar rests on bed rock with no tie connectors. Each superstructure is composed of a thin reinforced arch shell, shaped like an inverted catenary. This slab is 5 in. thick at the crown and 7 in. at the springing line (Fig. 1).

The shell is stiffened and supported by reinforced concrete arch ribs 25 ft on center which project above the shell thus providing a smooth interior roof surface unbroken by any obstructions. The ribs vary from 5 to 7 ft in depth and from 20 to 24 in. in width. The design was adopted to provide maximum fire resistance and minimum form cost. The hangar arches have a clear span of 340 ft, a clear center height of 90

*Presented at the ACI New England Regional Meeting, Boston, Mass., Nov. 10, 1949. Title No. 46-25 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 6, Feb. 1950, *Proceedings*, V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Chief of Engineering Division, U. S. Engineer Office, New England Division, Boston, Mass.

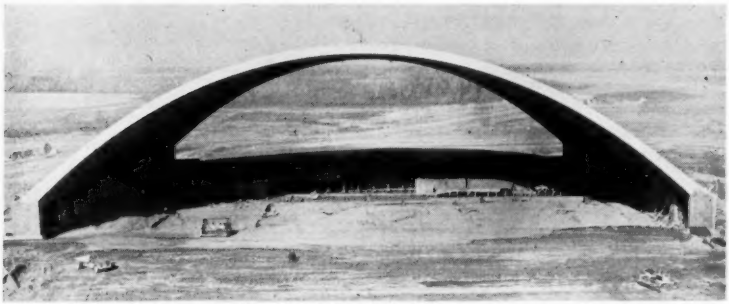

Fig. 1—End view of hangar

ft, and a length of 300 ft. Contraction joints are provided every 50 ft longitudinally.

An arch rib is located on each side of a contraction joint and at the midpoint between joints. The ribs are supported on reinforced abutments which project to the springing line of the arches. The arch contraction joints extend through the abutments to the floor at the third points of the hangar, and to the tops of the abutments at the other alternate bays.

The natural foundation at the Limestone site was a glacial till overlying a limestone rock at an average depth of 22 ft below the hangar floor. Drill explorations indicated considerable variations in the bearing value of the till so it was decided to found the substructure on rock and to eliminate tie rods between opposite piers.

CONSTRUCTION

Each abutment of the substructure was placed in three lifts, referred to as footings, pedestals and columns. The footings are 8 x 35 ft in plan, the maximum height being 7 ft. The largest plan dimensions of the battered pedestals below the hangar floor are 4 x 26½ ft, the height being dependent on the elevation of rock. The columns are 3 x 14 ft and extend 16 ft above the hangar floor to the springing lines of the arch ribs. The columns are tied together at the top by a slab and longitudinal beams at the springing line.

Freezing temperatures usually occur for about seven months in the year at Limestone, allowing a very short normal working season for pouring exposed concrete. Therefore, it was deemed advisable to place the substructure during the fall and winter months to allow as much favorable weather as possible for the superstructure; the footings being much easier to protect against the elements.

Foundation excavation

Foundation excavation was started in September, 1947. Continuous trenches were excavated along each side of the hangar for the footings. This procedure was less costly than individual excavation and sheeting for each footing and provided better opportunity to remove disintegrated surface rock.

Winter construction

Concrete was placed at a minimum temperature of 60 F. Cold weather protection of the footings and pedestals consisted of a complete housing for each unit, heated by means of gasoline-fired hot air heaters. Heat was supplied for 2 to 3 days prior to concreting and temperatures within the inclosures were maintained well above the required 50 F for the specified 72-hour period following concrete placement. The housing remained in place for the entire curing period with heat supplied only as necessary to prevent freezing. No concrete was frozen although the atmospheric temperature was as low as −30 F.

Construction of the columns started in April, 1948, after which time only limited protection was necessary. The abutments (Fig. 2) were completed in July, 1948, and required approximately 4900 cu yd of concrete and 412 tons of steel.

Superstructure falsework

The construction of the arch superstructure involved several problems not commonly encountered in concrete buildings. The arch centering consisted of a heavy timber falsework on which adjustable trusses were erected to carry the final forming. Wheels were installed at the base of the falsework to enable the entire centering to be moved longitudinally on 12 parallel, equally spaced railroad tracks. The falsework was made of 12 bents braced by means of horizontal trusses. The vertical members of each bent were 6 x 6 in. or 8 x 8 in. timbers. With few exceptions, the diagonal members were 2 x 6 in. and the horizontal

Fig. 2—Arch abutments

members, 3 x 6 in. All joints where two or more members connected to the same face of a post were bolted with ½-in. bolts; all other joints were well spiked. The centering was raised by 96 rail jacks and blocked and wedged in position at each post of the bents. Final adjustment of the form was made by 176 15-ton jacks under posts of the adjustable truss at the top of the falsework. Fig. 3 shows a view of the falsework.

The plywood sheathing of the form was directly supported by 2 x 10-in. joists and purlins. The centering structure included an area sufficient to place a 50-ft long unit and required about 280,000 fbm of lumber. Erection of the centering was initiated in April, 1948, and completed the first week of June. The first unit of the roof was placed on June 16-17, and the last unit on September 29, 1948. Generally about 3 weeks elapsed between successive pours, which time included the decentering, movement of the falsework, centering and placement of reinforcement and other embedded items. Minor patching of surfaces and placing of ceiling light supports were accomplished as the form was moved ahead.

Aggregate processing

Aggregate for the concrete was processed from a pit developed at the

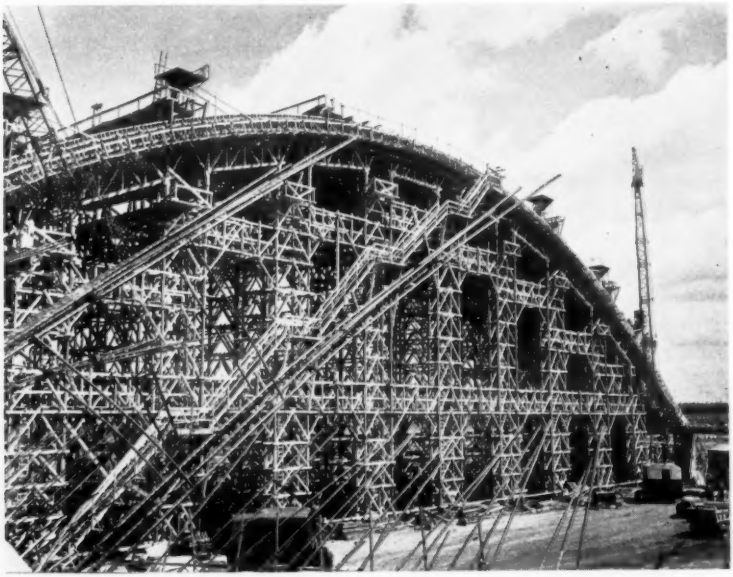

Fig. 3—Superstructure falsework

job site. Extensive tests were made on materials from this source prior to any concrete construction and it was determined that suitable concrete aggregates could be made from the gravel with proper processing. Other potential sources were investigated but the economies of producing local sand and gravel dominated in the decision to utilize this material. The pit from which the sand and gravel was obtained was a glacial outwash deposit containing a mixture of sandstone, limestone, quartzite and shale particles. The raw material contained an average of 10 percent of +3-in. material, about 55 percent of −3-in. to + No. 4 material, and 35 percent − No. 4 material. The deposit was highly stratified and contained silt and clay sizes in appreciable quantity.

Material was excavated in the dry by shovel and truck-hauled to a crushing, screening and washing plant located about 1500 ft from the pit. Two difficulties were encountered in processing this material: (1) the adequate washing of gravel and (2) the production of sand to meet the specification requirements. The coarse aggregate was generally coated with a tenacious layer of silt and clay which at times necessitated the rewashing of these sizes. Grading requirements of the sand made it necessary to excavate selectively in the pit. Materials produced consisted of 1½ to ¾-in. coarse aggregate, ¾-in. to No. 4 coarse aggregate and sand. As the aggregates were produced, they were truck-hauled as required about 2 miles to the batching plant, and the material surplus to current demands was stock-piled adjacent to the processing plant.

Mixing

As concrete for the substructure was started in late November, 1947, the batching plant was entirely housed and heated. The aggremeter was of the conventional 3-compartment bin type. Batching of the fine and coarse aggregates was accomplished in a 1 yd weigh batcher, the weighing operation being performed cumulatively by means of a triple arm, horizontal beam type scale. For winter operation, the aggregates were heated in the stockpiles by covering with tarpaulins and introducing warm air supplied by gasoline-burning heaters. In addition, steam pipes were placed around the hopper of the aggremeter. A hand operated dispenser for the air-entraining admixture was installed for discharging the admixture on to the sand in the weigh batcher. Mixing water was heated to 100-120 F, as necessary, to obtain the desired concrete placement temperature. During the early part of the work, bag cement was utilized, but soon after the start of concrete operations facilities for unloading and storing bulk cement were installed.

Concrete was conveyed from the batching plant to the job site and mixed by two methods. Three 3-cu yd end-charge transit mix trucks were utilized as more or less stand-by units while 1-cu yd pavers were

more generally used. The transit mixers were charged with water and aggregate, then moved to the cement loading plant for cement. Mixing was accomplished en route to the site of operations. When using the pavers, multi-compartment dump trucks hauled the dry-batched materials and discharged their load in 1-yd lots to the mixer skip.

Placing concrete

Chutes conveyed the concrete from the mixing unit to the footing forms. The round bottom chutes were set up to effect deposition of the concrete at five points along the long axis of the footing. With this setup, it was possible to use mixing units at both ends of the footing. The concrete for the footings was readily handled and consolidated with a 2 to 3-in. slump. Air entrainment produced a very cohesive mixture readily consolidated by vibration and practically eliminated bleeding. The concrete was sufficiently cohesive to leave the chutes in a ribbon equal in thickness to the depth of chute and after passing through hoppers and elephant trunks would build up in a mound with little evidence of segregation. Concrete for the pedestals and columns was handled similarly except that it was hoisted in buckets to hoppers at the top of the form.

The first roof pour of the hangar was started at 6:12 A.M. on June 16, 1948, and the last bucket of concrete was discharged at 9:48 A.M. on June 17, a total elapsed time of 27 hours and 36 minutes. This time was shortened considerably as the workmen gained experience, the shortest time of the 600-cu yd placement being 15 hours and 20 minutes. Both pavers and transit mix tracks were utilized in this operation. Two cranes with 100-ft booms and 20-ft jibs hoisted the concrete in 1-cu yd buckets from the mixing units at ground level to hoppers serving buggy runs at various elevations.

At the top of the arch, buggy runs reaching approximately 75 ft of the form transversely were installed. Three runs were erected on each side of the arch at about equal intervals from the top run. Each buggy run on the slope had five outlets leading to sectional chutes of semi-circular cross section. Three of the outlets served the arch ribs and two served the intermediate bays of the shell. The buggy runs at the top of the arch were erected so that concrete could be discharged directly from the buggies to the form. Concrete was placed progressively from the springing line to the top of the arch, the sides being placed simultaneously to prevent undue stresses in the arch centering which might be caused by eccentric loading.

The ribs were placed first for a distance of about 10 ft along the arch (Fig. 4). In placing the ribs, care was exercised to work the concrete out a short distance from the bottom of the form to minimize the possi-

Fig. 4—Placing concrete on the hangar roof

bility of honeycomb in the slab area adjacent to the ribs. Placement in the ribs was followed continuously by placement of the slab in 8 to 10 ft stretches, this being an optimum distance for proper finishing. Contrary to what might be expected, it was necessary to place concrete at an average slump of 4 in. Concrete drier than about a 3-in. slump could not be properly worked at the base of the ribs or in the slab owing to the position and amount of reinforcing steel and the difficulty in proper vibration of the concrete in the thin shell. On the other hand, concrete of a consistency wetter than about a 4½-in. slump presented trouble because of the tendency to flow excessively.

As the concrete discharged from the chutes in the lower portion of the arch, it was traveling quite fast and an unrestricted discharge tended to segregate the concrete as well as to result in undesirable accumulation. This was remedied by building a baffle consisting of a 2-in. board to which two handles about 10 ft long were attached. As concrete accumulated behind the board, the baffle was allowed to move slowly down the slope with its load of concrete. It was necessary to maintain the finishing work immediately behind the placing operation because of the rapid setting of the concrete in which Type III or high early strength cement was used. As soon as the slab was screeded and wood floated, membrane curing compound was applied to the surface with a manually operated spray gun. Throughout each arch pour, observations were made to determine settlement and other movement of the form. Several sets of 6 x 12-in. test cylinders and 6 x 6 x 21-in. beams were fabricated during each pour and subsequently tested.

In addition, 4 x 6 in. x 4½ ft beams were fabricated for modulus of elasticity determinations performed by the architect-engineer representative on the job. The specifications required that the concrete

attain a modulus of elasticity, as measured by the above-mentioned beams, of 2,000,000 psi and, a compressive strength of 2500 psi as measured by the 6 x 12 in. cylinders prior to decentering. As a rule, these strengths were obtained in about 72 hours.

Floor slab

Placement of the door track foundations and the floor slab was started in September, 1948, and completed in December. Methods of concrete manufacture were similar to those described for other portions of the hangar. The distribution differed in that concrete was pumped from a pumpcrete machine located at the west side of the hangar to the point of deposition.

The floor slab for 100 ft along the center of the hangar, where heavy wheel loads can occur, is 24 in. thick. Adjacent slabs 100 ft wide on either side of the central section are 18 in. thick. Repair shops and utility rooms occupy the outer 34 ft along each side of the hangar and the floors in these areas are of nominal depth. The heavier slabs within the central 300-ft square rest on a very compact gravel fill. Contraction joints, ¾ in. wide, are located around the perimeter of this square and along the quadrant lines. Construction joints occur at 20-ft intervals longitudinally and at 40, 50 and 60-ft spacing transversely. The central slabs are heavily reinforced and the contraction joints are doweled with 2-in. round pipes, 18 in. on center. The floor was poured in alternate sections to minimize shrinkage. An integral finish ½ in. thick was placed on all floors before the underlying concrete had set.

CONCRETE MIXTURES

The concrete mixtures utilized were designed on the job, being based on specification requirements and adaptability of the mixture to placing requirements for any particular form. All concrete test specimens, with the exception of those made during the early stages of construction, were tested in a job laboratory adequately equipped for such testing. A summary of concrete requirements and test results is given in Table 1.

All concrete with the exception of the roof construction contained Type II modified heat of hydration cement. High early Type III cement was used in the roof construction. An air-entraining agent was added at the batching plant to all concrete mixtures. The total volume of concrete in the completed structure was 16,000 cu yd.

INTERIOR DETAILS

Radiant heating coils were installed throughout the main hangar floor. The radiant heat is supplemented by unit heaters located near the large doors and by fin-type radiators in the utility rooms on either side of the

TABLE 1—CONCRETE DATA

Item	Footings	Columns	Roof	Floor
Specification requirements:				
Min. cement factor, bags per cu yd	5.5	6.0	6.0	6.0
Max. W/C ratio, gal. per bag	7.0	6.0	6.0	5.5
Slump, in.	1½–4	3½–6	2–6	1½–4
Compressive strength, psi				
Min. ave. 5 cylinders	3000	3750	3750	4500
Min. any one cylinder	2400	3000	3000	3600
Modulus of rupture, psi	—	—	—	700
Concrete mix data:				
Cement per cu yd batch, lb	557	592	611	564
Sand per cu yd batch, lb	971	1092	1248	1078
Stone per cu yd batch, lb	2193	1956	1736	2008
W/C ratio, gal. per bag	5.4	5.25	5.5	5.0
Cement factor, bags per cu yd	5.9	6.3	6.5	6.0
Ratio sand to total aggregate, percent	34	36	42	35
Max. size aggregate, in.	1½	¾	¾	1½
Slump, in.	2–3	3–4	4	1½–2½
Strength data (Ave.)				
7-day compressive strength, psi	2505	2996	4806	3476
28-day compressive strength, psi	3574	4830	5860	5116
90-day compressive strength, psi	5353	5505	—	—
7-day modulus of rupture, psi	—	—	—	499
28-day modulus of rupture, psi	—	—	—	700

hangar. Heat will be supplied from a high-pressure hot water central boiler plant. A heat exchanger located in the hangar permits the use of low pressure hot water in the heating system in the building.

The interior of the structure (Fig. 5) provides an unobstructed hangar area 300 ft square. Floor drains are of sufficient capacity to care for normal operations and to handle a fire sprinkler discharge of 19,300 gal. per minute. Lighting fixtures suspended from the arch ceiling provide 25 ft-candles of light over the hangar area.

The roof is insulated with 1-in. thick fiber glass over which is laid a built-up roofing of four plies of perforated asbestos felt and asphalt.

The hangar doors (Fig. 6) are notable for their size. When open they provide a clear entrance 40 ft high and 300 ft wide at each end of the

Fig. 5—Interior view of completed hangar

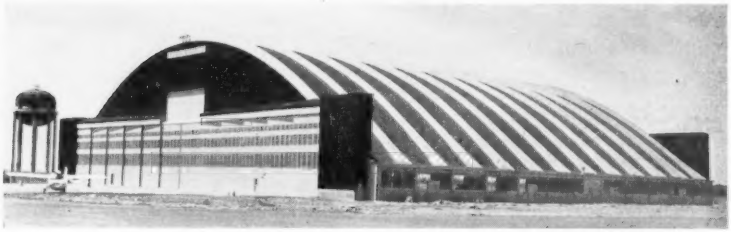

Fig. 6—Exterior view of completed hangar

structure. Six doors, each 40 ft high and 25 ft wide nest into an enclosure on either side of the opening. These units travel on rails embedded in the sill and are controlled simultaneously by motor-driven cable operation. A vertical lift tail gate 20 x 50 ft provides additional clearance at the center for larger planes. All doors are of insulated steel construction. About 4 minutes are required to open the doors to the full clearance.

The spandrel areas between the large doors and the arch roof, and the walls of the door pockets are covered with asbestos protected steel supported on structural steel framing. The wall panels are insulated with 1-in. thick fiber glass to reduce heat losses.

ACKNOWLEDGMENT

The hangar was designed by Roberts and Schaefer Co. of Chicago. The basic design of the arch structure has not been included in this paper since the subject has been covered in numerous published articles. The Limestone hangar was built by the Lane Construction Corp. and T. W. Cunningham Co., Inc., under the supervision of the New England Division, Corps of Engineers, Dept. of the Army.

Discussion of a paper by John E. Allen:

Construction of Long-Span Concrete Arch Hangar at Limestone Air Force Base*

By ANTON TEDESKO†

The author describes clearly and concisely the construction of long-span shell-type U. S. Air Force hangars. A previous article‡ on the design of these hangars contains information on design details, dimensions, loads, etc. No published papers, however, have stated the reasons for the choice of the structural system nor have they dealt with the problem of optimum arch slenderness, a subject of increasing interest to structural engineers. It is the purpose of this discussion to outline the designing engineers' intentions and the economical aspects involved in the shaping of these hangars.

The requirements of the U. S. Air Force made it necessary to provide for 340 x 300 ft unobstructed floor space. This favored reinforced concrete as an economical solution because the hangar structure could be divided into a sufficient number of construction units to permit effective re-use of formwork. The possibility of constructing identical future hangars at the same site, and re-used form centering and equipment available from earlier constructions indicated further savings over the cost of hangars in other construction media.

The fact that concrete is economical for a member which is predominantly in compression made an arch the logical choice for a concrete hangar. Although the enclosed volume due to the rise of the arch exceeds minimum requirements for operating clearances, the small penalties incurred for heating and lighting cost did not outweigh the advantage of the low cost structural system.

A substantial number of "pilot designs" were made and priced. Span-rise ratio, arch-column stiffness ratio, and arch spacing were systematically varied to study their effect on economy. Current material and labor prices and well organized construction methods used by contractors on similar jobs formed the basis for the cost estimates. These estimates reflected the relative over-all advantages of one scheme with respect to another.

*ACI JOURNAL, Feb. 1950, *Proc.* V. 46, p. 405. Disc. 46-25 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Engineer-Manager, Structural Dept., Roberts and Schaefer Co., Chicago, Ill.

‡Prentiss, Louis W., "Thin Concrete Arch Roof Provides 340-ft Clear Span for Bomber Hangar," *Civil Engineering*, Feb. 1949, p. 34.

The results of theoretical analyses of arch ribs and thin shells may sometimes indicate to the designer immediate advantages or savings in reinforcing steel and concrete. Such apparent advantages were critically reviewed in the light of the over-all cost picture. Some resulting slight penalties were consciously accepted, because these disadvantages would be overcome by simplification in construction leading to over-all economy. The final design resulted in a rather slender arch, shaped to follow the thrust line for dead load, and supported on stiff columns. The adopted structural frame, embodying an almost completely restrained arch, is the most economical, considering both the functional requirements and the prevailing prices.

In an arch there is an intimate relationship between the safety factor for buckling and the magnitude of bending moments due to deformations. If the loads are an appreciable percentage of those which would cause buckling, the effective bending resistance of the arch rib will be reduced, as secondary moments due to plastic flow will increase rapidly.* On the other hand, the moments due to rib shortening, volume change or movement of the supports increase with increasing arch stiffness. However, the secondary moments due to deformations caused by the above effects, increase at an increasing rate as arch stiffness and buckling safety is reduced. From these few considerations it may be seen that a compromise has to be made in choosing the stiffness of the rib, and that a full evaluation of the buckling danger due to deformation stresses leads to stiffer arches than would otherwise be required. It is evidently due to this realization that foreign codes specify a minimum arch slenderness.†

A comparatively slender arch may not have the inherent strength to resist the high localized temperatures and pressures which might occur from fire and explosions such as were withstood in the 1945 Wright Field disaster.‡ Recognition of the possibilities of buckling due to induced vibrations caused by earthquake or wind forces may influence the choice of the factor of safety against buckling. A slender arch may require strict control of plastic flow. This can be attained by increased duration of curing and delayed decentering, which, however, is generally impractical because quick form re-use is an economic necessity. A smaller safety factor for buckling may increase the cost of cold weather concreting when the rate of strength increase of the concrete is low; this may delay the moving of the form centering and interfere with the contractor's progress schedule. With full consideration of the stated problems and the choice of a practical concrete strength, the designers dimensioned the arch so as to achieve optimum economy with the minimum acceptable safety.

*Dischinger, Franz, "Untersuchungen über die Knicksicherheit etc.," "Elastische und Plastische Verformungen etc.," *Der Bauingenieur*, 1937, p. 487-621; 1939, p. 53-572.

†Such as a limited arch rib stiffness ratio of $\frac{L}{d}$, wherein L represents the wave length for buckling of the arch and d the average depth of an arch of rectangular section of equivalent stiffness. (See also the ACI code requirements for long columns: $\frac{L}{d} \leq 20$.) L equals 121 ft for the hangars under discussion.

‡Tedesko, Anton, "Low-Cost Repairs Restore Concrete Hangar to Design Strength," *Civil Engineering*, Jan. 1947.

For a given arch rib cross section the arch stability is affected by the location of the connection of the shell slab with respect to the arch. By framing the shell at either the top or the bottom, the section becomes a "T" or an "inverted T," thus insuring maximum moment of inertia for the section. This is important to the designer because the buckling safety of the arch rib is directly proportional to the moment of inertia of the arch cross section.

Functional and economic consideration usually govern the position of the shell with respect to the arch ribs. For example, the interior shell surface of a penstock must present a minimum obstruction to the flow. On the other hand, in case of a submarine the ribs are placed on the inside of the shell in order to gain a streamlined body. For the hangars discussed the shell was connected to the bottom of the arch ribs. This achieves the simplest form movement and increases the stiffness through the resulting inverted T-section. The flange contributed by the shell reduces the compression steel necessary in the arch ribs. The interior roof surface of the hangar is unbroken by obstructions.

The concepts presented herein and the designing engineers' intention to obtain optimum stiffness (rather than minimum stiffness) with the least amount of material are in contradiction to a thesis recently advanced that the shell of such a structure should be placed at the mid-height of an arch rib, thereby indicating emphasis on minimum stiffness. Cost estimates were presented to support such theories; however, these estimates are not realistic as a comparison with the project discussed herein indicates. The bid price for the Rapid City hangar was $1,343,000, or $12.16 per sq ft including utilities.* This is much less than the cost one would obtain from the data presented in support of the mid-height location of the shell.

A 12 ft, 1:30 scale, model of the hangar described by the author is now being tested at Fritz Engineering Laboratory, Lehigh University. Results from this test will soon become available; preliminary research results indicate a close correlation between the theory used in the design of the subject hangars and model performance.

*Radiant heating coils throughout the main hangar floor were added at a cost of 54 cents per sq ft. A price addition negotiated several months after the contract award and not included herein arose from the decision to move the hangar to a different location in which the soil conditions required changes in the foundation system, including the addition of the piles and tie rods referred to in the author's paper.

Volume-temperature relationships were observed for air-dry, vacuum-saturated and partially saturated specimens to determine the

Volume Changes in Small Concrete Cylinders During Freezing and Thawing*

By RUDOLPH C. VALORE, JR.†

SYNOPSIS

The volume-temperature behavior of small concrete cylinders was observed, using a new mercury-displacement dilatometer, during freezing and thawing cycles in which the range 40 to −20 F was traversed at various rates. Specimens cast from a mix containing 6 bags of portland cement per cu yd, plain and modified by the addition of an air-entraining agent, were tested in air-dry, vacuum-saturated and partially saturated conditions.

Volume-temperature relationships for air-dry specimens yielded uniform thermal expansion data, but those for vacuum-saturated specimens showed departures in the form of transient expansions during freezing, and residual expansions following thawing of about 0.8 and 0.4 percent, respectively; a single slow cycle produced decreases in dynamic E exceeding 60 percent.

Much smaller departures were observed for partially saturated specimens (65 to 85 percent of vacuum-saturation) and included, in addition to transient and residual expansions, secondary effects of freezing termed "shrinkage" and relaxation phenomena. The magnitude of the transient and residual expansions appeared to depend upon the rate of cooling and heating, the degree of saturation of the specimen and, during the slow cycle, upon the degree of supercooling before freezing.

INTRODUCTION

This paper describes the exploratory phase of an investigation of the physical reactions occurring in concrete specimens in various degrees of moisture saturation during several kinds of freezing and thawing cycles. The purpose of this exploratory phase was to observe the volume-temperature relationships for air-dry, vacuum-saturated, and partially saturated specimens within the range 40 to −20 F.

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The results presented herein are condensed from National Bureau of Standards Research Paper RP 2000 which contains detailed information on the materials, procedures, and the construction and calibration of equipment used in the study.[1]

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Thomas,[2] in England, investigating the freezing of building stones, found a dilatometric method unreliable for measuring changes in bulk volume (*i.e.*, volume of solids plus volume of pores) in incompletely saturated specimens, because the displacement medium, petroleum ether, entered to an undetermined extent the pore spaces not containing water. Thomas obtained interesting and valuable data by resorting to an optical lever extensometer method. It appeared that a dilatometer in which a hydrocarbon liquid was used as a displacement medium would prove unsatisfactory for the present study. Because of several advantages (including a relatively low thermal expansion coefficient and a high density) mercury was chosen as the displacement medium and steel dilatometers were constructed with which it was possible to measure the residual, as well as the transient, changes in bulk volume in concrete specimens during freezing and thawing.

EQUIPMENT AND PROCEDURES

Fig. 1 is a drawing of a mercury-displacement dilatometer containing a concrete specimen and immersed in the coolant bath. The dilatometer chamber was constructed from steel plate and tubing, with 1, 2, and 5-ml capacity measuring pipettes used in fabricating the detachable burette assemblies. Readings were estimated to 0.001 ml for the 1 and 2 ml

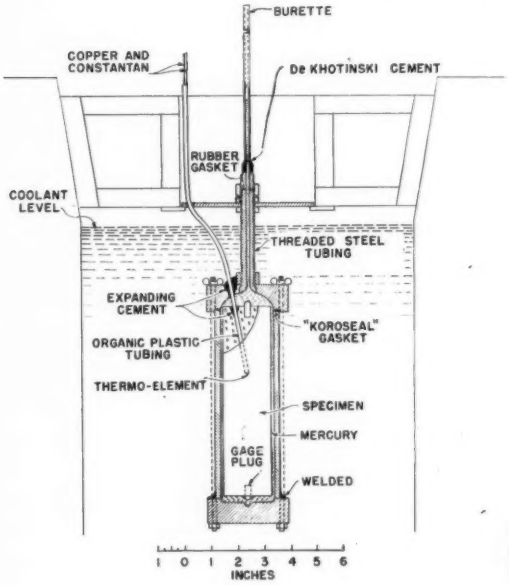

Fig. 1—Mercury-displacement dilatometer containing a small concrete cylinder

burettes, which were used for all tests except those of vacuum-saturated specimens.

A solid steel cylindrical "specimen," for which the thermal expansion characteristics were determined independently and having dimensions similar to those of the concrete specimens, was used to calibrate the dilatometer. The calibration, described in detail elsewhere,[1] provided a value for the thermal coefficient of volume change for each dilatometer chamber. This value and the known value for the thermal coefficient of cubical expansion for mercury permitted the calculation of corrections which were applied to the burette readings to yield the actual change in volume of a specimen at any temperature during a test. All observations were made under conditions of temperature equilibrium, which, for the purposes of the present investigation, were assumed to exist for a maximum temperature difference of 0.5 F between the center of the specimen and the coolant in which the dilatometer was immersed.

The test specimens were cylinders, 1.92 in. in diameter and 7.20 in. long, cast from a concrete containing 6 bags of portland cement per cu yd, Potomac River sand and $\frac{3}{8}$-in. gravel. The basic mix proportions were, by weight, 1 cement: 2.8 sand: 2.5 gravel, on the basis of saturated, surface-dry aggregate. Actually, dry aggregates were used. Air entrainment was effected in half the mixes by the addition of 0.01 percent, by weight of cement, of a proprietary resin in neutralized solution. Gravimetrically determined air contents were 2.5 ± 0.5 percent for the plain, and 5.3 ± 0.6 for the aerated mix. Net water-cement weight ratios were 0.56 for the plain and 0.53 for the aerated mix. The bulk volume of each specimen was 338 ± 3 cu cm.

The surfaces of the specimens were rendered impermeable to mercury by rubbing a 1:1 cement-fine sand mortar into the surfaces so as to fill the exposed voids. This treatment did not appreciably impede the passage of air and water during various curing operations.

All specimens whose behavior is described in this paper were cured in a fog room at 70 F to the age of 28 days prior to further curing of various kinds which will be described later.

The tests consisted mainly in subjecting each specimen, while in a dilatometer, to a number of freezing and thawing cycles of different kinds within a single temperature range, 40 to −20 F. In this way, the effects of different rates of cooling and heating were observed, unobscured by the unpredictable dissimilarities that often occur in "duplicate" specimens. The cycles included a slow cycle, requiring 32 hours or longer, in which the temperature was lowered or raised stepwise at a rate not exceeding 10 F per hour,* and a 1 hour fast cycle, in which the

*Initially, the slow cycle consisted of 8 hour cooling and heating periods separated by 16 hours at −20 F. Because of limitations of equipment, however, the bath temperature could, in many cases, be brought only to a temperature near −10 F. during the first 8 hours, decreasing gradually from this point to −20 F during the next 16 or more hours.

dilatometer, initially at 40 F, was transferred to a −20 F bath and 30 minutes later was retransferred to a 40 F bath. An interrupted fast cycle differed from the 1 hour cycle in that the dilatometer remained in the −20 F bath for 16 hours or longer between 30-minute cooling and heating periods. Other cycles were used in which cooling and heating at different rates were combined.

The "percent of vacuum saturation" was used as a measure of the degree of saturation and was calculated as $(W_I - W_O/W_S - W_O) \times 100$, in which W_I is the weight of the specimen as removed from the dilatometer after testing, W_O is the weight after 24 hours oven drying at 230 F, and W_S is the weight after 7 days immersion following saturation with distilled water after evacuation for 24 hours at 4 mm of mercury. Obviously, the percent of vacuum saturation would equal the true percent of saturation only when the treatment produced complete saturation. It can be shown that the percent of vacuum saturation so obtained represents an upper limit for the true percent of saturation.

A measure of the mechanical changes produced in each specimen by the freezing and thawing was obtained in terms of changes in Young's modulus calculated from the fundamental frequency of flexural vibration before and after the dilatometer tests.

RESULTS AND DISCUSSION

Air-dry specimens

Volume-temperature relationships for a plain and an aerated specimen,* air-dried for 125 days at 70 F and 50 percent relative humidity following the initial moist curing, are shown in Fig. 2 for both a slow and a fast cycle. Values for the thermal coefficient of linear expansion, calculated as one-third of the slopes of the straight-line relationships shown in the figure, are given in Table 1, and do not vary significantly with the type of cycle. The dilatometers met the requirements of Meyers for satisfactory determinations of thermal expansion of cement pastes, mortars and concretes, *i.e.*, "airtight containers which do not restrict the movement of the specimens."[3]

Vacuum-saturated specimens

The specimens whose behavior is shown in Fig. 2 were vacuum-saturated following the testing in the air-dry condition, and were then stored in water for one year. Fig. 3 shows the volume behavior observed during one very slow cycle of freezing and thawing. Similar behavior was observed for other vacuum-saturated specimens.

The specimens contracted uniformly when cooled from 40 to approximately + 25 F, at which point expansion, attributable to the

*Specimen numbers are prefixed by the letter P for plain specimens, and A for those containing the air-entraining agent.

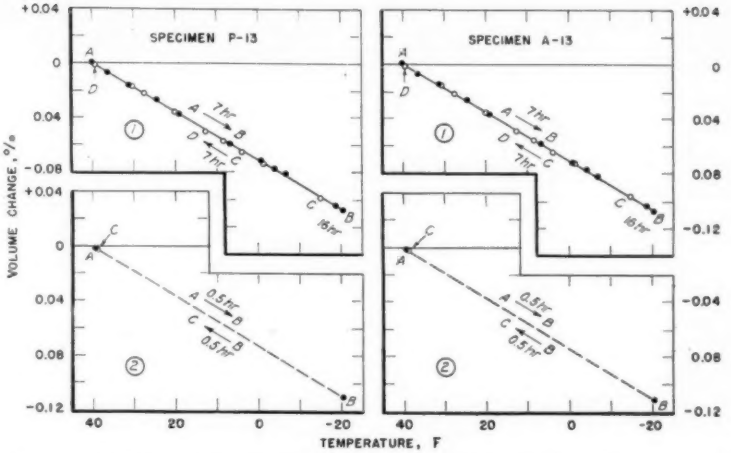

Fig. 2—Volume-temperature relationships for slow and fast cycles for air-dry specimens, P-13 (plain) and A-13 (aerated). Thermal coefficients of linear expansion calculated from these data are given in Table 1. ●—cooling, ○—heating.

TABLE 1—THERMAL COEFFICIENTS OF EXPANSION FOR TYPICAL AIR-DRY SPECIMENS CALCULATED FROM DATA SHOWN IN FIG. 2

Specimen	Temperature change	Thermal coefficient of expansion	
		Cubical per deg F	Linear* per deg F
P-13	slow cooling	17.7×10^{-6}	5.9×10^{-6}
	slow heating	17.9	6.0
	fast cooling	18.1	6.0
	fast heating	17.6	5.9
A-13	slow cooling	17.7	5.9
	slow heating	17.9	6.0
	fast cooling	17.9	6.0
	fast heating	17.8	5.9

*Calculated as 1/3 of cubical coefficient.

freezing of water within the pore systems, began and continued for about 30 minutes. During this time, while the bath temperature was held at + 25 F, the temperature of the specimens first increased to about 31 F, and then returned to equilibrium with the bath, indicating a "supercooling" effect.* When both the volume and the temperature became stable, point C was observed. The resumption of cooling pro-

*"Supercooling" as herein used is defined as the cooling of a specimen below the apparent freezing point of the contained water without thereby inducing freezing, as indicated by these volume changes.

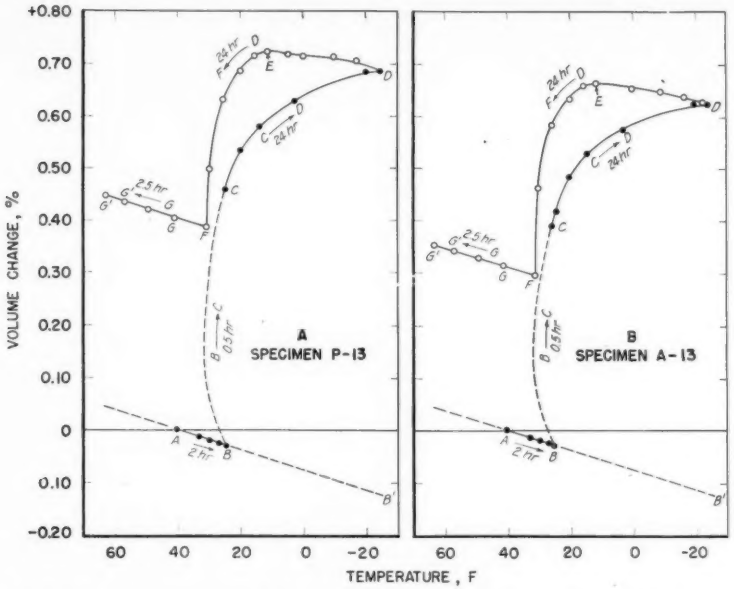

Fig. 3—Volume-temperature relationships for a slow cycle for specimens P-13 (plain) and A-13 (aerated), tested in the vacuum-saturated condition.

Specimen P-13 decreased 65 percent in dynamic E, and specimen A-13, 60 percent as a result of this cycle. Data obtained in tests of these specimens in the air-dry condition are shown in Fig. 2. •—cooling; ◦—heating.

duced further increases in volume, and at point D (−25 F) the specimens had not begun to contract.

Heating produced first, additional increases in volume, and then, beginning at approximately +15 F, contractions which were attributed to the melting of ice within the pores of the specimens. The final melting point, or the temperature above which no ice remained in the specimens, appeared to occur at 31 F, since further heating produced uniform increases in volume to point G.

Maximum volume expansions (indicated by the distance DB′ at −25 F) were 0.80 percent for the plain, and 0.75 percent for the aerated specimen; residual expansions (at 40 F) were 0.40 percent for the plain and 0.31 percent for the aerated specimen. Residual increases in length, measured independently, were 0.12 percent for the plain and 0.11 percent for the aerated specimen and are in good agreement with values obtained by dividing the residual increases in volume by three.

There were few visible indications of deterioration in the specimens and these were limited to small spalls or popouts.* Young's modulus, however, was affected as follows:

	Specimen P-13	Specimen A-13
Dynamic E, before testing, psi	6.3×10^6	6.3×10^6
Dynamic E, 7 days after testing, psi	2.2×10^6	2.5×10^6
Change in dynamic E, percent	−65	−60

After storage in water for one year, the specimens had recovered approximately three fourths of the loss in dynamic E. The decrease in dynamic E immediately following the dilatometer testing was estimated at about 90 percent,[4] indicating, for practical purposes, the almost complete destruction resulting from one cycle of freezing and thawing.

The high degree of saturation of these vacuum-saturated specimens is not likely to be attained in masses of concrete exposed to natural weathering nor, as will appear in the following section, even in small specimens subjected to prolonged immersion. A fairly complete saturation may occur, however, in upper layers of horizontally exposed surfaces. Volume changes of the magnitude shown in Fig. 3 occurring in these layers could then so injure the surface as to promote scaling.

Partially saturated specimens

Volume-temperature behavior for various types of cycles

The volume-temperature behavior described thus far is typical of specimens representing extremes in saturation. Most of the specimens tested, however, were in a partially saturated condition. This group includes all of the moist specimens for which artificial means of saturation (evacuation or high pressure) were not used, regardless of the duration of curing. Volume-temperature relationships for three specimens subjected to various types of cycles are shown in Fig. 4 to 6. A-27 (an aerated specimen) is typical of a relatively low, and P-23 of a relatively high degree of saturation, while P-27 is intermediate between these extremes for partially saturated specimens.

The "0" ordinate for each graph represents the base volume for each specimen, taken at 40 F at the start of the first cycle. The displacement of the starting points for successive cycles away from the "0" ordinate indicate a cumulative residual change in volume—usually an expansion—produced by the various cycles.

*These specimens were recently air-dried at room temperature after storage in water for more than two years following the testing in the dilatometers. A network of irregular lines, lighter in color than the background, was visible over the entire surface of each specimen after drying. The lines were apparently indicative of cracking caused by the freezing and thawing cycle; viewed through a microscope, the light-colored deposit appeared to have healed the cracks completely.

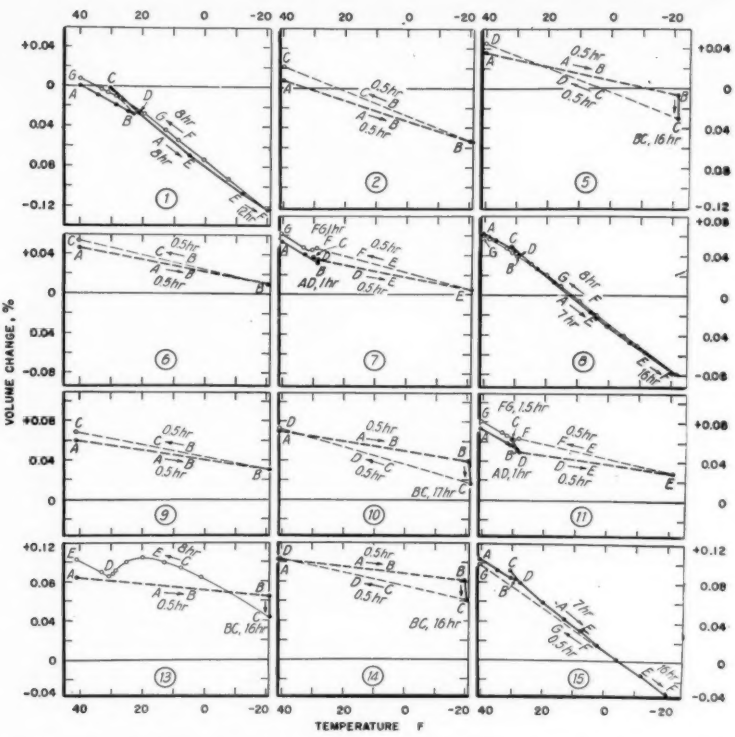

Fig. 4—Volume-temperature relationships for 12 cycles of various types for specimen P-27 (plain, 75 percent of vacuum saturation).

Cycles 1 and 8 were slow cycles; 2, 5, 6, 9, 10, and 14 were one-hour and interrupted fast cycles; 7, 11, 13, and 15 were combined cycles of several kinds; and 3, 4, and 12 were moderate cycles which are omitted from the present discussion. The specimen was stored in water at 70 F for 60 days and was 90 days old when tests began. The specimen decreased 27 percent in dynamic E during 15 cycles. •—cooling; ○—heating.

The slow cycle—General characteristics of the volume behavior of partially saturated specimens during the slow cycle are shown in Fig. 7. (Note also cycle 1 in Fig. 4 to 6.) P-11 was one year old, and had been air-dried for 60 days and immersed for 270 days prior to testing; P-26 was 37 days old and had been immersed for one week. The specimens were approximately equal in degree of saturation: 78 percent of vacuum saturation for P-11 and 77 percent for P-26.

When cooled slowly (10 F or less per hour) the specimens showed uniform contraction to temperatures as low as 21.5 F (AB in Fig. 7).

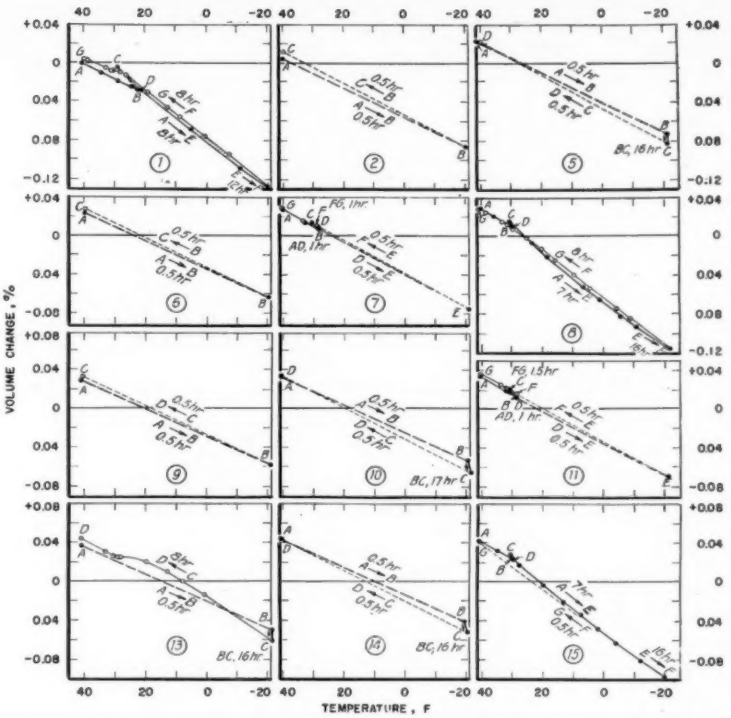

Fig. 5—Volume-temperature relationships for 12 cycles of various types for specimen A-27 (aerated, 67 percent of vacuum saturation).

The testing schedule, curing and age of this specimen were the same as those for specimen P-27 (Fig. 4). A-27 decreased 11 percent in dynamic E during 15 cycles of freezing and thawing. •—cooling; ○—heating.

At B an abrupt increase in temperature and volume occurred, presumably as water within the pore structure of each specimen began to freeze, and maximum values, C, were attained within a few seconds. Following the reestablishment of temperature equilibrium between each specimen and the bath, further cooling produced continuous, but not always uniform, contraction. The heating curves, as shown, were usually displaced upward from the cooling curves; with the apparent final melting point of ice in the pore structures (30 to 32 F) there was only a gradual change in slope (Fig. 7 B) or a discontinuity (7 A), less sharply defined than during freezing.

Thermal contraction and "shrinkage" during slow cooling—For most specimens (those less than 100 days old) the slow cycle behavior was

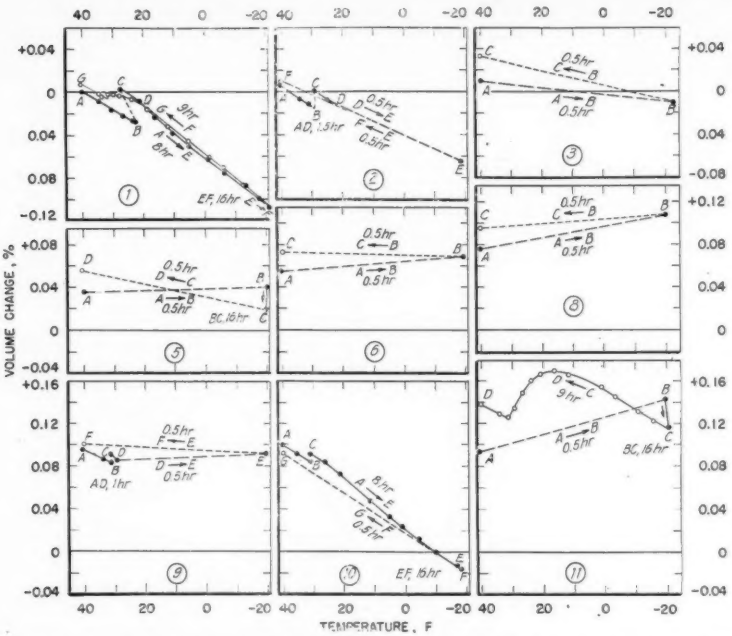

Fig. 6—Volume-temperature relationships for 9 cycles of various types for specimen P-23 (plain, 85 percent of vacuum saturation).

Cycle 1 was a slow cycle; 3, 5, 6, and 8 were one-hour and interrupted fast cycles; 2, 9, 10, and 11 were combined cycles of several kinds; and 4 and 7 were moderate cycles, which are omitted. The specimen was stored in water at 70 F for 49 days and was 80 days old when tests began. P-23 decreased 34 percent in dynamic E during 11 cycles and was the highest in degree of saturation of all partially saturated specimens tested. •—cooling; ○—heating.

typified by 7 B. It is interesting to note the difference in slope of the portions of the cooling curve preceding and following point C. Assuming that AB extrapolated to B′ would represent the contraction had freezing not occurred, it may be seen that the actual contraction was greater, since CD intersects and descends below AB′. The total volume change during slow cooling then appears to consist not only of the normal thermal contraction, upon which the expansion at C is superimposed, but also of a "shrinkage" factor. This phenomenon occurred as an effect of freezing, having been absent in the behavior observed for air-dry specimens. Thomas[2] observed extra-thermal decreases in length in partially saturated sandstones following freezing which were ascribed to internal "drying" occurring as a consequence of the "removal" of water converted to ice in previously unfilled pore spaces. The fact that

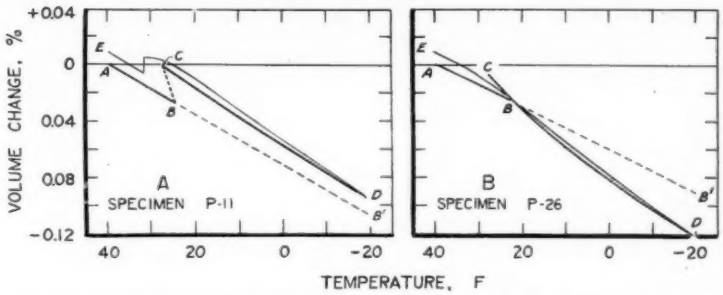

Fig. 7—Volume-temperature relationships for a slow cycle for virgin specimens P-11 (plain, 78 percent of vacuum saturation) and P-26 (plain, 77 percent of vacuum saturation).

P-11 was air-dried for 60 days at 70 F and 50 percent relative humidity, stored in water at 70 F for 270 days, and was one year old when tests began. P-26 was stored in water at 70 F for 7 days and was 37 days old when tests began. P-11 decreased 37 percent in dynamic E during 8 cycles of various kinds, and P-26 decreased 19 percent during 18 similarly varied cycles.

the "shrinkage" effect was much reduced or not detected for some of the older specimens, such as P-11 (Fig. 7 A), may be related to physical changes in the cement paste structure resulting from prolonged hydration.

Repeated slow cooling—In Fig. 8 relationships for repetitions of the slow cooling are plotted to a common origin for each specimen so that they might be compared with the slow cooling behavior shown for the first cycle for each specimen. The data are taken from Fig. 4 to 6. In each case the specimen was supercooled to a lower temperature before freezing during the first cycle than during subsequent slow coolings. Fig. 9 portrays the behavior of a specimen subjected to 7 successive slow

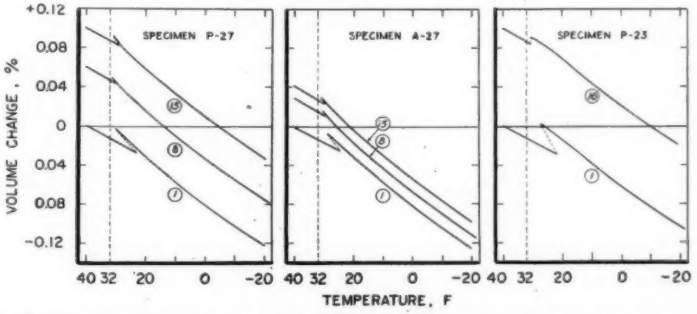

Fig. 8—Volume-temperature relationships, during repeated slow cooling, for three partially saturated specimens, differing in degree of saturation.

The data are taken from Fig. 4 to 6 and plotted to common origins. Percent of vacuum saturation: P-27, 75; A-27, 67; and P-23, 85.

cycles. The degree of supercooling (point B) for cycles 2, 4, and 7 is comparable with that for cycle 1; while in cycles 3, 5, and 6 supercooling was virtually absent. Fig. 8 and 9 show apparently contradictory behavior. It was found, however, that the effect shown in Fig. 8, in which the degree of supercooling diminished with repetitions of the slow cooling, did not occur during any slow cooling prior to which the specimen was maintained unfrozen for a relatively long period of time. Specimen P-29 (Fig. 9) was maintained at 40 F prior to cycles 2, 4, and 7 for periods of 111 to 136 hours, while prior to cycles 3, 5, and 6 the period of "recovery" did not exceed 17 hours. For all specimens so studied there was a "critical" period of about four days which, when exceeded, apparently caused the specimen to react to slow cooling in the same manner as during the first cycle. For slow cycles shown in Fig. 4 to 6 and 9, the magnitude of the abrupt expansion following supercooling and of the residual expansion for the complete cycle appeared to be related to the degree of supercooling.

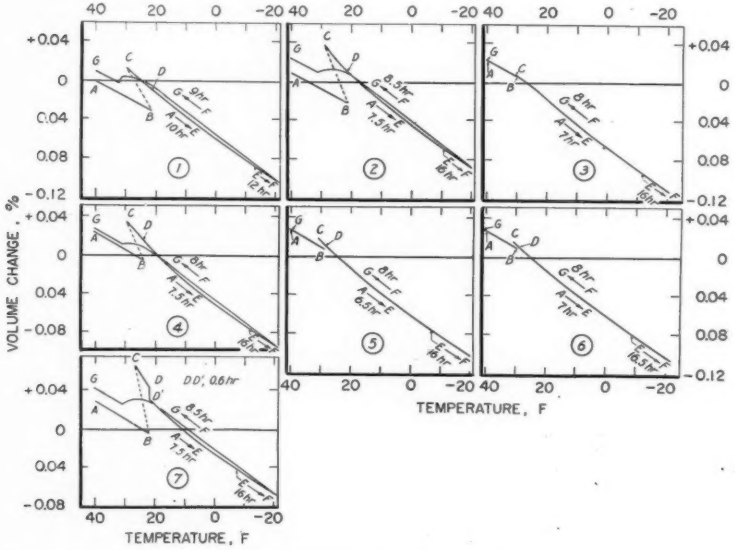

Fig. 9—Volume-temperature relationships for 7 successive slow cycles for specimen P-29 (plain, 81 percent of vacuum saturation) showing differences in supercooling.

Storage at 40 F prior to cycles 3, 5, and 6, approximately 16 hours, and prior to cycles 2, 4, and 7, 111 to 136 hours.

P-29 was stored in water at 70 F for 90 days and was 120 days old when tests began. The decrease in dynamic E during testing was 17 percent.

Fast cycles—Only three readings were taken for the 1-hour cycle. These indicate the transient volume change (at −20 F) resulting from cooling, and a residual expansion following heating (at 40 F). Fig. 4 to 6 and, more clearly, Fig. 10, show significant differences in contraction during the fast cooling, not only among the three specimens, but also, for specimens P-27 and P-23, among the several fast cycles given each specimen.

Cycles 5, 10, and 14 in Fig. 4 and 5, and cycle 5 in Fig. 6, were interrupted fast cycles. In each case there was a decrease in volume while the temperature of the specimen was maintained at −20 F for approximately 16 hours following the 30 minute cooling period. This contraction was found to occur at a gradually diminishing rate for some specimens held at −20 F for several days.

Combined cycles—Cycle 13 in Fig. 4 and 5, and cycle 11 in Fig. 6 were combinations of a fast cooling and a slow heating. A decrease in slope in the heating curve for this type of cycle occurred at a much lower temperature than in the identically obtained heating curves for the slow cycles (cycle 1 for each specimen); during cycle 11 specimen P-23 began to contract when heating progressed above approximately +16 F.

The combination of the slow cooling and the fast heating was also employed (cycle 15, Fig. 4 and 5, and cycle 10, Fig. 6). There was a residual *contraction*, rather than an expansion, at the completion of this cycle.

Also shown in Fig. 4 to 6 are relationships for cycles in which the cooling was slow only until equilibrium was attained following the abrupt discontinuity at point C, after which the remainder of the 40 to −20 F range was traversed in a single 30-minute step (cycles 7 and

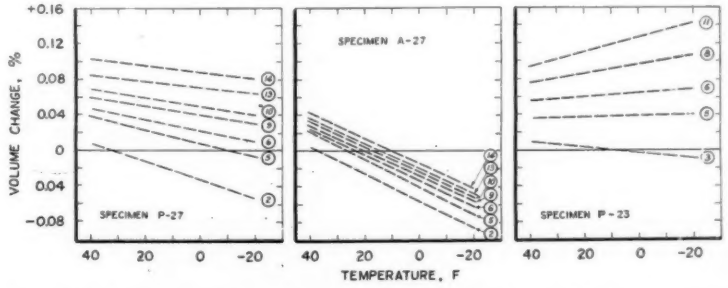

Fig. 10—Volume-temperature relationships, during repeated fast cooling, for three partially saturated specimens, differing in degree of saturation.

The data are taken from Fig. 4 to 6 and plotted to a common origin. P-27 and A-27, identical in age and curing history, were tested simultaneously in two dilatometers. Degrees of saturation are indicated under Fig. 8.

11, Fig. 4 and 5, and cycles 2 and 9, Fig. 6). Had all of the freezable water been frozen at point C, the contraction during the "slow-fast" cooling and during the slow cooling should have been approximately equal for each specimen; *i.e.*, increasing the rate of cooling below point C should have had no effect. Since the "slow-fast" cooling actually produced an appreciably smaller contraction, it appears that not all the freezable water froze at a single temperature.

Comparison of various types of cycles—It may be seen, in Fig. 4 to 6, and 9, that during slow cooling the volume of each specimen at any specified temperature below approximately +20 F was virtually an irreducible minimum, inasmuch as the cooling curve showed no appreciable alteration in slope during the final temperature decrement, EF, despite a much reduced rate of cooling. The slow cooling (when supercooling was a minimum) therefore apparently allowed a maximum degree of accommodation of the expansion of water upon freezing within the previously unfilled pores in the specimen.

In Fig. 11 data for each specimen for a sequence of different types of cooling (taken from Fig. 4 to 6) are plotted to an arbitrary common origin. The pattern of behavior shown was common to all partially saturated specimens tested, namely, the faster the rate of cooling, the smaller was the contraction during cooling.* Note that differences in contraction during the fast coolings starting and ending each sequence are small compared with differences between successive coolings of different kinds. Considering a smaller contraction as an indication of greater strain, or a smaller degree of accommodation, the previously described contraction at −20 F following cooling, observed during the

*Cooling portions for cycles obtained by traversing the 40 to −20 F range in 2, 3, or more equal temperature steps adhered to the pattern without exception. These "moderate" cycles are omitted from the present paper and account for gaps in sequences of cycle numbers in Fig. 4 to 6.

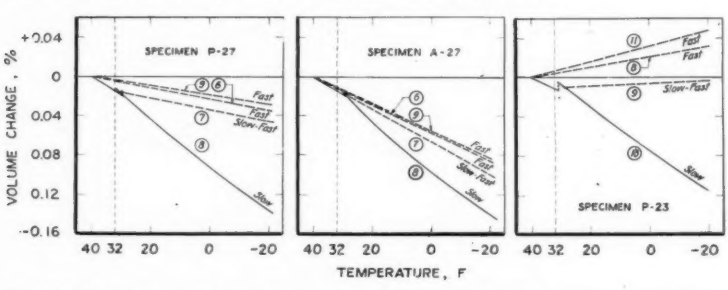

Fig. 11—Comparison of volume-temperature relationships for three types of cooling for three partially saturated specimens differing in degree of saturation.

The data are taken from Fig. 4 to 6 and plotted to an arbitrary origin. The sequence of cooling was fast, slow-fast, slow, and fast for each specimen. Degrees of saturation are indicated under Fig. 8.

TABLE 2—EFFECT OF FREEZING AND THAWING CYCLES UPON VOLUME AND ELASTICITY OF SPECIMENS FOR WHICH VOLUME-TEMPERATURE RELATIONSHIPS ARE PRESENTED IN FIG. 2 TO 11

Specimen*	Age, days	Condition	Percent of vacuum saturation	Number of cycles	Total residual volume change, percent	Change in Young's modulus (dynamic),† percent	Fig. in text
P-13	155	air-dry	15	5	0	−4	2A
A-13	155	air-dry	15	5	0	−4	2B
P-13	650	vac.-sat.	100	1	+0.400	−65	3A
A-13	650	vac.-sat.	100	1	+0.310	−60	3B
P-27	90	part. sat.	75	15	+0.100	−27	4,8,10,11
A-27	90	part. sat.	67	15	+0.040	−11	5,8,10,11
P-23	80	part. sat.	85	11	+0.135	−34	6,8,10,11
P-11	360	part. sat.	78	8‡	+0.085	−37	7A
P-26	37	part. sat.	77	18‡	+0.085	−19	7B
P-29	120	part. sat.	81	7	+0.040	−17	9

*Listed in order of appearance in text; prefix "P" refers to plain specimens and "A" to those containing air-entraining agent.

†Changes based upon measurements prior to start and 48 hours after completion of dilatometer tests, except for case of P-13 and A-13 (vac.-sat.) for which second determination was made one week after dilatometer tests. Changes shown for P-13 and A-13 (air-dry) may not indicate effects of testing, since specimens were in dilatometers for over 6 weeks.

‡Fig. 7 shows only first cycle.

interrupted fast cycle, might be considered as a decrease in strain, *i.e.*, a "relaxation" effect. Hypotheses of frost action presented variously by Thomas,[2] Kennedy,[5] and Powers[6] would then appear to account for some of the behavior of these specimens during cooling. These hypotheses ascribe the disruptive force of freezing to a hydraulic pressure within the pore structure produced by a movement of water ahead of an "ice front" and generated by the expansion accompanying freezing, rather than to a direct crystal pressure caused by ice formation.[6] Data on specimens from Fig. 2 to 11 are summarized in Table 2.

The "shrinkage" effect described for slow cooling should also occur during fast cooling, but it was not possible to differentiate between shrinkage and relaxation in the contraction at −20 F. Older specimens, however, such as P-11, which showed relatively small shrinkage effects during slow cooling (Fig. 7 A), also showed large contractions at −20 F following fast cooling.

On the basis of the hydraulic pressure hypothesis, there appear to be several measures of the strain imposed in a specimen by freezing. Of these measures, the transient strain (observed at −20 F), and the residual volume change resulting from the complete cycle, might also furnish an indication of the vulnerability of a given specimen to frost

action. Should further experimentation verify these implications, the possibility of predicting durability on the basis of the measurement of transient or residual strain for one or two cycles of freezing and thawing suggests itself.

Present observations indicate that the residual expansion may be a relatively insensitive measure of the reaction of a specimen to freezing. It has been shown that one type of cycle (slow cooling-fast heating) produced residual *contractions* in the specimens; yet, it is doubted that the specimens were actually improved by the treatment. While reasons may be advanced to show that slow thawing may be more destructive than fast thawing, the use of residual volume change as a sole criterion for vulnerability of concrete to freezing and thawing requires further study of the effects of different rates of thawing. With these reservations, the following ranking of some of the cycles, in the order of increasing residual expansions, is given: (1) slow cooling*—fast heating; (2) slow cooling*—slow heating; (3) fast cooling—fast heating and (4) fast cooling—slow heating.

Differences among specimens

References have been made to the observed differences in transient volume behavior among various specimens when subjected to a given type of cycle. The relationships in Fig. 11 typify the behavior of specimens differing in degree of saturation, but not appreciably in age or curing, while Fig. 7 shows typical behavior for specimens differing in age and curing, but not in degree of saturation. These examples, however, should not be used to oversimplify the behavior of all concrete since differences in cement, aggregate, etc. are not considered here.

Similarly, only a composite analysis of residual effects appears warranted. For 10 pairs of specimens, a plain and an aerated specimen comprising each pair, and both specimens in each pair having undergone the same series of different kinds of cycles, the analysis is as follows:

	Plain	Aerated
Average percent of vacuum saturation	78	69
Average percent change in dynamic E	−22	−13
Average percent total residual volume change	+0.092	+0.043

It is seen that the specimens containing an air-entraining agent were of an appreciably lower degree of saturation than the plain specimens. Among the specimens studied no differences in behavior were detected between plain and aerated specimens other than those that might be ascribed to differences in saturation.

*Supercooling absent or reduced to a minimum.

Specimen P-23 (Fig. 6) deserves additional comment, since it not only showed the smallest contraction during fast cooling of all specimens tested, but also showed the greatest change in contraction during repeated fast coolings, and even showed an appreciable expansion during the final cooling (cycle 11). The final two cycles (slow cooling-fast heating and fast cooling-slow heating) produced rather remarkable movements, and the specimen was considered to have "failed," having decreased 34 percent in dynamic E as a result of the 11 cycles of freezing and thawing. This specimen exceeded all other partially saturated specimens in degree of saturation—85 percent of vacuum saturation—but the movements shown during any one of the cycles were small in comparison with those exhibited by vacuum-saturated specimens (Fig. 3).

SUMMARY

Volume-temperature relationships for air-dry specimens were virtually straight lines whose slopes were the thermal coefficients of cubical expansion for the concrete. Reproducible values for the coefficients were obtained for each specimen, regardless of the rate of cooling or of heating.

Vacuum-saturated specimens showed departures from uniform behavior in the form of expansions during cooling (as large as 0.8 percent) which were about one-half compensated by contraction during heating so that there were relatively large resultant "residual" expansions. These specimens definitely "failed" (decreased in dynamic E 60 percent or more) during one very slow cycle of freezing and thawing.

Transient and residual expansions for partially saturated specimens (65 to 85 percent of vacuum-saturation) were very much smaller in magnitude than those observed for vacuum-saturated specimens and appeared to depend upon the degree of saturation, the rates of cooling and of heating, and, for the slow cycle, the degree of supercooling before freezing.

Secondary effects of freezing were "shrinkage" and relaxation phenomena which consisted of contractions in excess of those due to purely thermal causes and contractions at constant temperature following freezing.

There was evidence that water in the pores of the specimens supercooled before it froze during slow cooling, and that the degree of supercooling was influenced by the previous freezing and thawing history of each specimen. There was no evidence of supercooling during the fast cycles.

It appears that the measurement of transient volume changes *during* freezing and thawing may permit an estimate of the vulnerability of a specimen to frost action.

Differences between plain specimens and those containing an air-entraining agent appeared to be limited to the effects of rather marked differences in degree of saturation.

Such variables as brand or type of cement, type of aggregate and curing, were not included in the present study.

ACKNOWLEDGMENT

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Gratitude is also due Gerald Pickett of Kansas State College and T. C. Powers of the Portland Cement Assn. Research Laboratory for helpful discussion and comments during the progress of the study.

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Intensified efforts toward control of concrete production will pay dividends in terms of increased service life of concrete structures.

Inspection and Control of Concrete for Highway and Bridge Construction*

By H. W. RUSSELL†

SYNOPSIS

Inspection procedures and personnel assignments used by the Illinois Division of Highways for large, medium and small jobs are described. The importance of assurance that materials used are adequately tested and approved before shipment, determination of proper proportions by trial mixtures, control of air entrainment, correction of batch weights and proper preparation and testing of concrete specimens are emphasized. The inspection of ready-mixed concrete supplied to small jobs poses a problem still lacking an adequate solution.

The production of concrete is a process which will stand many abuses without immediate apparent detrimental effect. Years may pass before the structure begins to show evidence of such abuses, and by that time little may be remembered as to what actually took place. Unsatisfactory placing of concrete, on the other hand, is apt to become apparent upon the removal of the forms, or when cores are drilled from the concrete before acceptance of the work.

Since it is human nature to desire a completed job of satisfactory appearance, the primary efforts are too often exerted in that direction, and inspection of the production of concrete is apt to become nominal, unless enough personnel is assigned to handle this phase of the work. It is probable, therefore, that intensified efforts toward control of the concrete in general will pay greater dividends, in terms of increased service life of concrete construction, than improvements which at this time may be possible in other phases of concrete inspection. It follows also that such intensified efforts are not likely to be exerted unless an inspector, who will not be removed to perform other duties to the detri-

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ment of the control of the concrete mixture, is definitely made responsible for this work.

CONTROL OF CONCRETE MIXTURES

The Illinois Division of Highways assigns full time proportioning engineers to control the concrete mixtures in the construction of pavements and large structures. On smaller jobs, a part time proportioning engineer may be assigned, or the control of the production of concrete may be exercised by the resident engineer, who usually worked as a proportioning engineer at some time before attaining this position and is fully capable of performing this work.

On small jobs, and on some city and county sections requiring the supervision of the Illinois Division of Highways, it has in the past been difficult to obtain more than nominal inspection of the concrete mixture. A proportioning engineer in charge of a mobile testing unit (Fig. 1) in each highway district now visits each of these jobs as often as possible during the construction season, with apparently good results.

The most difficult control problem has been found in the case of ready-mix plants delivering concrete to highway construction, often in relatively small amounts and in intermittent loads. At present, the policy is to handle each case individually as may be most suitable. Where

Fig. 1—Mobile testing unit used by the Illinois Division of Highways. The following equipment is used in the field for the proportioning of concrete mixtures and field soil tests: (left to right) beam testing machine, specific gravity and moisture determinator and accessories, such as scoop, weigh pans and scale weights (This apparatus was developed by W. M. Dunagan to meet the requirements of engineers engaged in proportioning concrete materials by weight.), water bucket, slump can, cement sampler, air determinator for the determination of air in plastic concrete, shovel, aggregate sample on canvas, putty knife and steel wire brush, beam forms, aggregate sample splitter, dairy scale and aggregate bucket, dish pan and Coleman stove for drying samples, aggregate pans, rack containing sieve and brushes for cleaning sieves, calibrated ice cream can and platform scale for concrete yield determination, shovel, torsion balance and weight box, pycnometer and funnel apparatus for density test, two soil augers, small scoop, funnel and guide used in making density test, press for removing soil sample from Proctor mold, cone bearing machine, metal rammer for compacting soil, Proctor mold and wood block used for cleaning augers.

All of the equipment shown in the picture can be carried in the truck. All of the concrete proportioning equipment except the shovel, beam forms and beam testing machine fit neatly into a cabinet built inside the truck. There is not enough headroom in the truck to permit a man to work inside, although he can work on the truck bed from the rear end of the truck when the doors are open.

relatively large volumes of concrete are involved and the deliveries are reasonably continuous, a proportioning engineer is assigned.

The district engineer of materials in each highway district and his assistant have immediate supervision of the work of the proportioning engineers. The Bureau of Materials of the Illinois Division of Highways exercises general supervision through a general inspector who contacts not only the district engineers of materials but also the proportioning engineers on the individual jobs as often as possible during the construction season. These personal contacts with the engineers in the field have been found invaluable both in the solution of problems which arise in the field and in obtaining information for use in formulating policies and procedures.

The materials entering into the concrete generally have been inspected at the source, and it is one of the foremost duties of the proportioning engineer to assure himself that all shipments of aggregates and cement have been properly tested and approved, that they have arrived in good condition and that only inspected and approved materials are used in the concrete mixture. This, as a general rule, presents no difficulties in the ordinary case, but it may involve undue complications when concrete is delivered from ready-mix plants, especially in intermittent loads. Just how this problem should be handled at such plants is being given serious consideration at this time, but no definite policy has been established.

Before concrete operations start, the proportioning engineer checks the weighing devices and the water measuring devices for accuracy and for compliance with the specification requirements, which process may be repeated at any time during the progress of the work if inaccuracies develop. He watches the batching operations during the progress of the work as much as his time will permit, except that on the large jobs, someone else may be assigned to do this.

Usually, depending somewhat on the experience of the proportioning engineer, the district engineer of materials or his assistant is present at the beginning of concrete operations. The proportions of the materials are first determined by trial mixtures. This presents little difficulty, because practically all commercially available aggregates in Illinois have been used in similar construction over a number of years and it generally is possible to set the proportions on the basis of past experience. The process, therefore, may involve only a few minor adjustments during the mixing of the first few batches.

One problem which has caused some difficulty is that of obtaining the desired air content of the concrete. When air entrainment is obtained by the use of an air-entraining agent added during the process of

mixing the concrete, no difficulties in making the adjustments are ordinarily encountered. The use of air-entraining cement sometimes has resulted in the necessity for further addition of an air-entraining agent on the job to increase the entrained air, or for blending non-air-entraining cement with the air-entraining cement to decrease the entrained air.

The batch weights are determined so as to produce a yield of mixed concrete exactly equal to the guaranteed capacity of the mixer, assuming a saturated, surface dry condition of the aggregates. Since the aggregates usually are not surface dry, the proportioning engineer must correct the batch weights and the amount of mixing water added at the mixer for deviations of the aggregates from this condition. He is provided with equipment for rapid determinations of the free water held by the aggregates, or the absorption of water by the aggregate, as may be the case, on which to base these corrections. Similarly, he makes corrections of the batch weights for fine aggregate contained in the coarse aggregate and for coarse aggregate contained in the fine aggregate, the No. 4 sieve being the division between the two, so that the proportion of mortar to coarse aggregate may remain constant.

To exercise this control properly, the proportioning engineer must watch the concrete mixture as much as possible and must keep constantly informed as to the workability of the concrete and any minor adjustments made in the amount of mixing water, as well as to the air content of the concrete. The best procedure is to have the proportioning engineer make both the slump tests and the air content determinations on the same batch of concrete for direct comparison. Experience has established the desirability of making air content determinations at least each time the batching from a new car of cement is started and, in addition, as often as the duties of the proportioning engineer will permit, and when an immediate need for such determination may be indicated.

The importance of making these tests as a check of the accuracy of the batch weights, especially when the concrete contains entrained air, cannot be overemphasized. Visual inspection of air-entraining concrete may be extremely misleading, and the proportioning engineer may find himself permitting concrete of 4- or 5-in. slump, with perhaps excessive air content, to be placed when only 2- or 3-in. slump is desired, if he does not perform these tests. Material reduction in air content, on the other hand, may manifest itself immediately in reduced workability and an apparent need for upward adjustment of the amount of mixing water and, when this condition develops, an immediate check of the air content should be made.

The yield may be determined by summation of the absolute volumes of all ingredients of the concrete, including the air as determined by

tests. This method, however, assumes that these absolute volumes are in fact determined correctly and that the ingredients for the particular batch for which the air content determination was made were weighed or measured out correctly. More important in the control of the mixture is the yield test by the gravimetric method, which, if conducted carefully, is a valuable over-all check of the degree of correctness of all the phases of proportioning and control. The proportioning engineer is required to make this test, especially where apparent need for this over-all check exists. This test has to some degree fallen into disuse during the last few years because of the amount of equipment needed and because it is somewhat burdensome. The question of restoring it to its former prominence is being considered.

As a check of the quality of the concrete obtained, the proportioning engineer makes and tests concrete beam specimens in the field. These specimens when removed from the forms are cured in wet sand at a central location and tested at ages ranging from 3 to 14 days. Though it has been suggested that the beams should be cured on the site and in the same manner as the concrete which they represent, we believe that the strength record obtained by the method adopted reflects the efficiency of the control of the production of concrete more satisfactorily. We also believe that the record of beam tests reflects at least the potential quality of the particular construction represented by the beams, provided the placing and curing of the concrete is done properly.

The proportioning engineer is required to keep complete records of all tests and adjustments made by him, and to make daily reports to his district office and the Bureau of Materials. From time to time he also reports the results of the beam tests. These reports are checked carefully at the Bureau of Materials and, if errors or inconsistencies of consequence are found, they are called to the attention of the engineers in the field and to the general inspector, who may be in a position to visit the job in question.

At the end of the construction season, an analysis is made of the data contained in the reports. From correlations and comparisons of the data, it usually is possible to establish facts which are valuable additions to the general information on the subject. The analysis also serves as a check of the results obtained with respect to those obtained during previous seasons and is a general index of the effectiveness of the control exercised.

The daily reports of proportioning and the beam test reports from each job become a part of the permanent records held by the district office and are available in the event information on the proportioning and control of the concrete mixtures should become desirable at any time in the future.

In general, it may be concluded from the experience of the Illinois Division of Highways that accurate information as to just what was done and knowledge of the results obtained during any one construction season are important prerequisites for improvements during the next.

Title No. 46-28

Careful preparation is essential to reliable results from SR-4 gages. The authors tell how to do it in

Some Applications of Electric SR-4 Gages in Reinforced Concrete Research*

By EIVIND HOGNESTAD† and IVAN M. VIEST‡

SYNOPSIS

The writers describe some applications of SR-4 gages in reinforced concrete research at the University of Illinois. Special emphasis is given to the problems involved in waterproofing gages attached to steel which is later embedded in concrete.

INTRODUCTION

Electric SR-4 gages and their auxiliary instruments have been developed, improved, and accepted in a wide variety of applications within an amazingly short period of time. The basic principles involved in the operation of these gages are valid in all applications, but the details of their use must be modified in accordance with the characteristics of each type of material and each type of testing problem. Many such modifications have been made; and the techniques involved in measuring deformations of metals have been rather well established. SR-4 gages have also been used extensively in reinforced concrete research. However, due to the peculiarities of this material, such applications are still limited.

Electric SR-4 gages have been in use at the Talbot Laboratory of the University of Illinois in reinforced concrete research since 1944. Within this time the electric gages have played an important part in tests of footings, columns, and I-beam bridges with elements thereof. Strains have been measured in *short-time tests* on the surface of concrete as well as on steel embedded in the concrete. It is the purpose of this paper to present a general review of the methods involved and the results obtained.

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INSTALLATION OF GAGES

Outline of the use of SR-4 gages

As yet, SR-4 gages have been used at this laboratory only in short-time tests made in less than 4 hours. In two investigations, however, several such tests were performed with each specimen so that the gages remained in service for 3 to 5 weeks. But even so, no long-time stability was required as each test represented an independent unit based on separate zero readings. Thereby it is considerably easier to obtain a satisfactory service from the gages than in a field or long-time laboratory test. However, since some investigations were rather extensive in scope and large numbers of gages were used, the time and expense involved in their preparation had to be limited as much as possible without reducing the value of the strain readings.

The cheapest and quickest way of attaching gages to the reinforcing steel is to mount them, after the concrete has been cured, through openings provided in the concrete at the gage points by insertion of tapered wooden blocks. This method is used whenever the effects of the openings can be considered negligible. If about a week elapses between the end of the moist curing period and the attachment of gages, no difficulties caused by moisture are generally encountered in this manner.

In some cases, however, the gages must be mounted on steel before pouring since the errors introduced by inserting wooden blocks would be prohibitive. Many methods of waterproofing gages to be embedded in concrete have been reported.[1,3,6,7,8,9,12] Most of these methods are either not adaptable to the type of problems met at this laboratory or are too complicated and time consuming. The method used by the Illinois Division of Highways is, however, very similar to the present practice at this laboratory.[5,8]

Much less attention has been given to the method of attaching SR-4 gages to the surface of concrete. The heterogeneity of concrete and its internal moisture are two new factors which must be considered in addition to those met when working with steel. Due to the heterogeneous character of concrete, gages must be long enough to obtain representative results; the required length of the gage depends on the maximum aggregate size.[11]

Protection against concrete moisture could probably be achieved by the use of a moistureproof cement. However, it has been the experience of this laboratory that if the concrete is given a chance to dry for a week before attachment, no serious troubles need be expected due to moisture.

Gages on exposed steel

SR-4 gages are attached to steel in the conventional manner with the nitrocellulose DuPont Cement No. 5458 (Duco cement). First,

the particular location on the steel member is cleaned by grinding and polishing; in case of deformed reinforcing bars, some lugs are removed. As the gages are mounted after the concrete has been poured and cured, the cleaned areas on the steel are covered with a layer of wax to prevent rusting. Access to the steel is insured by tapered wooden blocks fastened to the steel. After the concrete has been cured and dried, the blocks and wax are removed, the steel surface is cleaned with acetone and the gages are cemented in place. The cement under the gages is allowed to dry for about 24 hours. It is desirable to heat the gages with heating lamps for at least 3 to 4 hours during the drying period to improve the drying and thus to increase the gage stability whenever possible. However, the elevated temperature may, in some cases, cause cracking of the surrounding concrete, and therefore the use of the heating lamps is not always advisable.

As a rule, gages which are attached after pouring and used immediately after they dry out, are not covered with any protective coat. However, if the specimens are tested several days or weeks after the attachment of gages, a protection against air moisture and mechanical damage is secured by a 1/16-in. coat of mycrocrystalline Petrosene wax. This coat is moistureproof only if the steel around the gages has been heated to the melting point of the wax prior to its application.

Gages on embedded steel

In this case, the electric gages are installed before the concrete is poured. They are cemented to the steel by means of Duco cement as soon as the steel surface has been cleaned. Then the gages are dried for about 24 hours, always including 3 to 4 hours of drying under heating lamps. Afterwards, an asphaltic coating designed for protection of gages and their connections embedded in concrete is applied and tested for impermeability.

The primary purpose of the protective coating is to waterproof the gages. The waterproofing procedure described below meets the following requirements:

1. The gages should be sufficiently protected to remain in working condition after pouring and curing. Thus the protective coat must be waterproof and be able to withstand abrasion and vibration during pouring as well as shrinkage pressures during setting and curing. The resistance to ground should remain larger than 20 megohms.

2. The area covered with waterproofing should be as small as possible in order not to alter the bond conditions more than necessary.

3. The procedure should be simple enough to be carried out at small expense and without special skill.

The material used for the protection of SR-4 gages embedded in concrete is Petrolastic Asphalt No. 155 manufactured by the Stancal Asphalt

and Bitumulus Co. Some of its properties are: flash point—425 F min., melting point—155-165 F, and ash—1.0 percent max. No. 18 Flamenol wire with $\frac{1}{32}$-in. insulation manufactured by the General Electric Co. is used for connecting wires. This wire is both heat and flame resistant and waterproof.

The Petrolastic asphalt is applied in two layers. It is important that the gage shows no leakage to ground and that the steel surface be free of any foreign substances at the time of the application of the first layer of asphalt. Any loose, water absorbing or soluble substance extending from the edge of the protective coat to the gage is likely to form a path for moisture and thus cause trouble. The steel surface should be heated well above the melting point of the waterproofing material to insure an impermeable connection between the steel and the asphaltic coat. Gages thus prepared and ready for application of the waterproofing material, are shown in the upper left corner of Fig. 1 on reinforcement for a column and in the upper part of Fig. 2 on a shear connector made from a 4-in. rolled steel channel.

After the steel has been cleaned and heated, the gage is covered with a thin coat of asphalt heated to a fluid state. As the steel is at about the same temperature as the waterproofing material, they bind well together. A good bond between the steel and asphalt is further insured by remelting the edges of the asphalt layer with a hot metal bar. After the asphalt has cooled and hardened, Flamenol lead wires are soldered to each gage wire. The middle gage in Fig. 1 represents this stage. Gages

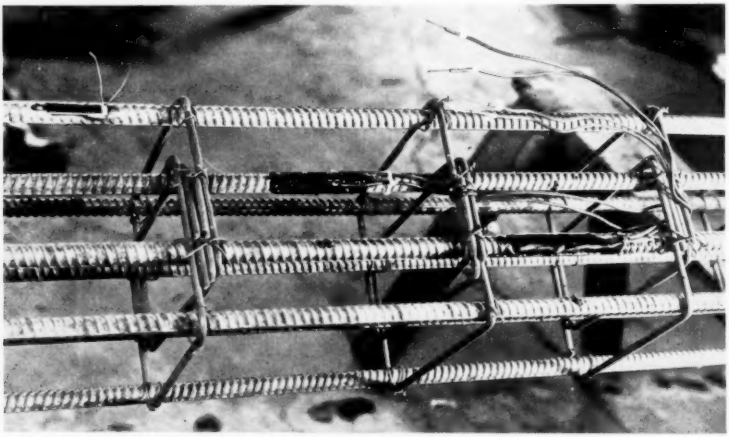

Fig. 1—Waterproofing of SR-4 gages attached to reinforcing bars

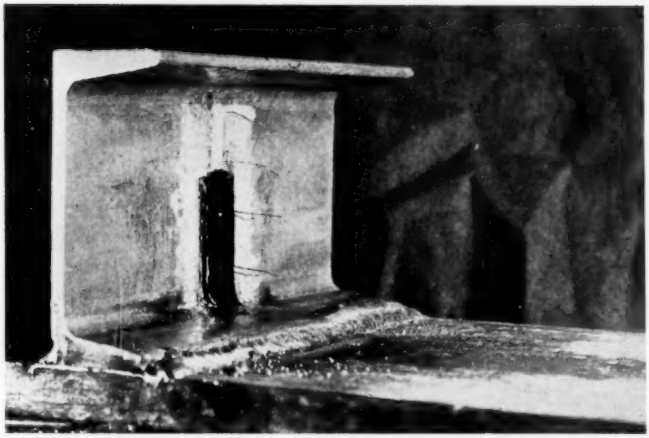

Fig. 2—Waterproofing of SR-4 gages attached to steel channel, first coat applied to two lower gages

covered with the first coat of asphalt are shown also in the lower part of Fig. 2.

While the primary task of the first coat is to waterproof the gage itself, the waterproofing of the soldered connection and a protection against mechanical damage is insured by a second coat. This layer is applied after slight heating of the first coat and must be sufficiently thick to insulate the soldered connections. Then, the waterproofing coat is trimmed to a final shape and its sides are remelted to close any possible pin-holes through the coat. The total thickness, width and length of the waterproofing coats vary from case to case, depending on the size and number of gages covered. For instance, four A-7 gages (¼ in. gage length) with one common ground wire and four separate lead wires are covered with asphaltic coating 1 in. wide and approximately ⅜ in. thick as shown in Fig. 3. The finished waterproofing of a gage on a reinforcing bar is shown on the right side of Fig. 1.

The waterproofing is tested by immersion in water for 12 hours. This test proved very useful as it reveals practically all cases of poor coating which result, as a rule, from imperfect bond between the steel and asphalt due to poor workmanship.

The waterproofing procedure may be summarized as follows:

1. Clean and heat the steel surface around the gage.
2. Apply a thin coat of asphalt around the gage and remelt its sides to insure good bond between the steel and the protective coat.
3. Solder wire connections.

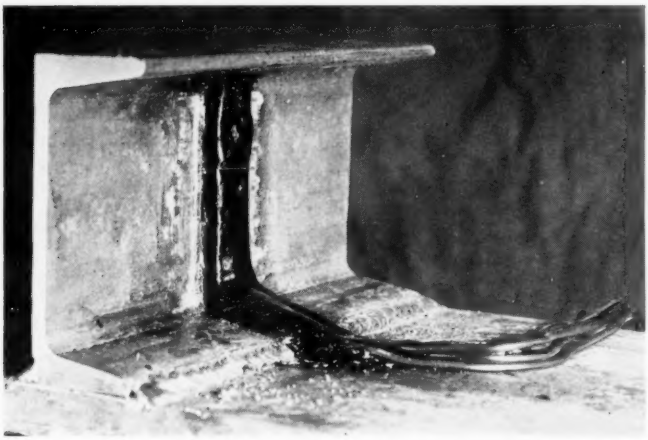

Fig. 3—Waterproofing of SR-4 gages attached to steel channel, final arrangement

4. After slight heating of the first layer apply a second coat of asphalt so as to cover all soldered connections.

5. Trim the protective coat to a final shape and remelt its sides to close up all holes in the protective coat.

6. Test the protective coat for impermeability.

As there is no way of replacing the bad gages after the concrete has been poured, the installation and waterproofing of the gages must be made with care and thought.

Gages on concrete

The procedure for attaching SR-4 gages to a concrete surface is similar to that for steel. After the concrete has been dried in air for at least a week, the location of the gage is smoothed and thoroughly cleaned with emery cloth and wiped off with acetone. Then, thinned Duco cement is worked into its pores to provide a continuous, uninterrupted base for the gage and protect the gage, to some extent, against concrete moisture. This coat is allowed to dry for 24 hours. The gage is cemented with a new layer of Duco and kept pressed against the concrete for about 12 hours. The drying is done in the air of the laboratory. Heating lamps are not used as in a few instances cracking of concrete due to uneven heating was observed.

As in the case of gages attached to exposed steel, no protective coat is provided unless the gage will be in use for a long time. In such cases a protective coat of Petrosene wax is used primarily for protection against mechanical damage as it is applied to a cold or only slightly heated concrete surface.

TEST RESULTS

The described methods of attachment and waterproofing of electric gages were developed primarily through five investigations in which over 4000 electric SR-4 gages of various types were used to measure both tensile and compressive strains in the reinforcing steel as well as on the concrete surface. The great majority of gages gave satisfactory service and consistent readings. A total failure occurred in only a few instances and was always traceable to improper workmanship. The most common types of failure were slipping of the gage and short to ground. While in the first case, the gage was naturally lost for further testing, satisfactory readings often were obtained in the case of a short to ground after a separate circuit had been formed.

Wall and column footings

This series of tests, which involved 24 wall and 132 column footings, was started in 1944. Since the footings were supported on a bed of heavy springs it was not possible to provide access to the reinforcement mat during testing, and the common mechanical strain gages could therefore not be used. Thus, SR-4 electric gages were used for the first time at this laboratory. The ¼-in. A-7 gages were used on ⅜-in. reinforcing bars and 1-in. A-11 gages on all larger bars.

All gages were cemented to the bars after the concrete had hardened and dried for about a week. Since the surrounding concrete was stressed in tension, only a negligible disturbance was caused by the wooden blocks used to provide access holes. The results obtained were very consistent.[14] Altogether, about 2500 gages were used. Shorts to ground could be checked and repaired before testing, and the great majority of gages did not start slipping before a considerable yielding had taken place.

Continuous I-beam bridges

Tests of 3 two-span continuous I-beam bridges were conducted in 1946 to 1948. All three bridges were one-quarter-scale models and consisted of 5 rolled steel I-beams and a slab of sand-cement mortar. Several short-time tests were made with each specimen. The strains in the ⅛-in. square reinforcing bars were measured by SR-4 gages, Type A-12 and A-7, with a gage length of 1 in. and ¼ in. respectively. The tensile strains in the mortar slabs were measured by A-11 gages. A total number of 431 gages were attached to the reinforcing bars and 30 to the slabs.

All gages were placed after the slabs were cast. As the gages had to remain in service for 3 to 5 weeks, each gage was covered with a protective coat of wax. At first, heating lamps were used to dry the gages; but their use was discontinued when cracks due to temperature stresses

were detected in the mortar slab. The over-all performance of the gages was very good. The dependability of their performance when attached to the mortar slab surface was confirmed by comparison with the strains measured in the steel bars and by results of tests of two control mortar beams. Some difficulties were experienced at the beginning of the test due to insufficient bond between the slab and the gages caused by lack of experience in attachment of gages.

Push-out tests of shear connectors

SR-4 strain gages embedded in concrete were used at this laboratory for the first time in push-out tests of channel shear connectors for composite T-beams. This series of tests has been in progress since February 1947. A total of 25 specimens with 196 electric strain gages attached to the channel shear connectors embedded in concrete slabs have been tested (see Fig. 2 and 3). All except 6 gages were Type A-7.

These tests represent a case where the gages must be attached before pouring concrete. The previously described technique of waterproofing was developed during the early stages of the push-out tests. From the last 107 gages (only these were attached by the method described), 97 gages met all three requirements, 8 gages showed low resistance to ground but gave useful readings, and 2 gages ceased to work entirely. Only in one case did the moisture enter under the protective coat of asphalt to the gages.

Composite T-beams

Tests of full-size composite T-beams began in December 1948. Up to the present date, 3 beams have been tested. Each beam consisted of a rolled steel beam with a concrete slab resting on the top flange. The concrete slabs and the steel beams were connected by channel shear connectors welded to the beam and embedded in the concrete of the slab. A total of 128 SR-4 Type A-7 gages were attached to the shear connectors, waterproofed and embedded in the concrete. Forty-seven A-9 gages, with a 6-in. gage length, were placed on the tension and compression surfaces of the slabs.

Several short-time tests were performed on each specimen and the gages were in service for a period of 3 to 4 weeks. Therefore, all gages attached to the concrete surface were protected against damage by a wax coating. No heat was used in connection with the concrete gages. In several cases, a short to ground was developed during the testing period, but all readings of the gages both on the shear connectors and on the slabs gave consistent results. Only one gage embedded in concrete gave erratic readings at higher loads and one gage on the surface of concrete ceased to function at the end of the third week of testing.

Eccentrically loaded columns

This series of tests has been in progress since February 1949. The 120 columns involved are subjected to eccentric loads parallel to the longitudinal axis. Strains are measured in steel as well as in concrete by means of electric SR-4 gages.

Strains on the concrete surface are measured by means of A-9 gages. In the tension zone, these gages are generally destroyed when a crack, visible to the naked eye, opens through the gage. In the compression zone, however, the gages function well all the way up to the ultimate load. To date, about 500 A-9 gages have been used. Measurements of strains in the reinforcement are made with A-11 gages. On the tension side, they are attached through core holes after the concrete has hardened. The gages on the compression steel and on all steel of 30 round, spiral columns, however, are embedded in the concrete. For the 90 square columns, 180 gages were embedded in concrete, all of which gave satisfactory service in spite of heavy abrasion and vibration while the concrete was compacted with a form vibrator. Another 270 gages were attached through core holes. Neither heating nor protective wax was used in these tests except for the embedded gages which were waterproofed as outlined.

CONCLUSIONS

Techniques of the application of electric strain gages in reinforced concrete research have been developed and their reliability proved by numerous tests. They permit the use of well-known advantages of the electric gages for measurements of strains on the concrete surface as well as in steel members embedded in concrete.

These techniques are, however, subject to limitations. First of all, they have been successfully applied only to short-time laboratory tests. Further, the waterproofing of gages to be embedded in concrete requires a very careful, though simple, workmanship.

A further simplification of the methods described and their extension to long-time and field tests is desirable.

ACKNOWLEDGMENTS

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Professor of Engineering Materials. Acknowledgment is also given to all present and past members of the research staff at the University of Illinois, who during their work on the investigations described, contributed to the solution of the given problems.

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Experimental evidence has established some of the shortcomings in conventional design methods. The authors propose formulas for determining the moments of resistance and the critical percentages of reinforcement.

The Ultimate Strength of Reinforced Concrete Beams*

By S. D. LASH† and J. W. BRISON‡

SYNOPSIS

The results of tests to failure on 57 small reinforced concrete beams are presented, and it is shown that they can be explained satisfactorily on the basis of a simple plastic theory of design. Formulas are proposed for determining the moments of resistance and the critical percentages of reinforcement.

INTRODUCTION

It is more important to know the load-carrying capacity of a structure, than it is to know the unit stresses produced by working loads. This principle has received increasing recognition during the past decade, and its application to reinforced concrete beams has been considered by a number of authors including, among others, Saliger,[1] Whitney,[2] Jensen,[3] Cox,[4] Hajnal-Konyi[5] and Guerrin.[6] Each writer has presented test results and has sought to interpret them by various more or less unconventional theories of beam action. This paper covers similar ground. The test results reported on a wide range of materials can be explained satisfactorily by a simple theory of plastic action. The authors' justification for following a well-trodden path is that the subject is obviously an important one and a large volume of experimental evidence is required to establish clearly the shortcomings of conventional design methods. The test procedure was unusual to the extent that loads were applied by dead weight rather than by means of an ordinary testing machine. This ensured that the load was not reduced in any way by the deflection of the beams.

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Although the test results apply to different qualities of steel and concrete, little information is given on beams reinforced with less than one percent of steel. In such beams the tensile strength of the concrete contributes an appreciable amount to the resisting moment of the member. It is hoped to make a separate study of these later. Also, the tests were made on small specimens, and there may be some question as to the applicability of the results to beams of other dimensions. The indications are that the scale effect is comparatively unimportant, but this matter also requires further investigation.

THEORY

According to the conventional or elastic theory, it is possible to calculate the bending moment that can be applied safely to a beam by selecting proper working stresses. The factor of safety involved in this procedure is considered to be the ratio of some limiting value of stress (usually yield strength for steel and cylinder strength for concrete) to the working stresses. On the other hand, the ultimate strength approach is concerned only with the maximum bending moment that the beam can resist, and the working load is determined by dividing this ultimate moment by a suitable factor of safety. The equations used in analysis and design by conventional methods are well known and need not be given here.

A reinforced concrete beam may fail either as a result of excessive stress or strain in the steel, or excessive stress or strain in the concrete. In theory, it is conceivable that both types of failure might occur simultaneously; if this happens, the beam is said to have a balanced section, and the percentage of reinforcement is referred to as the "critical percentage." It is obviously desirable to know the value of this critical percentage so that compression failures in the concrete can be avoided. Estimates of its value depend greatly on the theoretical basis used to determine it. According to the conventional theory it is usually about 1 percent, according to Jensen about 3 percent, and according to Guerrin 5 percent or more. These figures are very approximate, but indicate clearly the wide differences between the results given by different theoretical approaches. It is not easy to determine this critical percentage by means of tests, for reasons that will be explained later, but it can be stated quite definitely that the value is much greater than that given by the conventional theory.

PLASTIC THEORY

The plastic theory assumes that a beam will fail when either the steel or the concrete reaches a limiting value of strain, and it is further assumed that the distribution of stress on the compression side of the

beam can be represented without serious loss of accuracy by a rectangular block (see Fig. 1). If the steel has a well defined yield point, then the yield stress may be substituted for the limiting value of strain. If the steel does not have a definite yield, but has a stress-strain curve with a reasonably sharp knee, then the limiting strain may be replaced by a limiting stress, such as the yield strength at a given offset. For under-reinforced beams, therefore, it is necessary only to balance the strength of the steel (based on the yield point or yield strength) against the ultimate strength of the concrete. This leads to the following formula: (see Fig. 1)

$$\text{The total force in steel} = f_y\, pbd$$

$$\text{Total force in concrete} = f'_c\, bx$$

$$f_y\, pbd = f'_c\, bx$$

$$\frac{x}{d} = \frac{pf_y}{f'_c}$$

$$\text{Also } M = f_y\, pbd\left(d - \frac{x}{2}\right)$$

$$\text{hence } \frac{M}{bd^2} = f_y\, p\left(1 - \frac{pf_y}{2f'_c}\right)$$

This formula has been proposed by a number of authors, notably Saliger.[1]

While this formula is rather simple, it should be noted that it is a quadratic equation in p and consequently the design of a beam will also

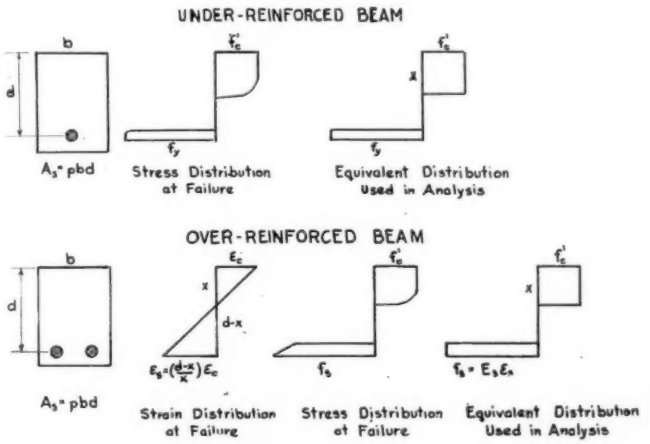

Fig. 1—Assumed stresses and strains in beams at failure

require the solution of a quadratic equation. As an alternative, the following linear equations may be used with small loss of accuracy:

For structural grade reinforcement	(f_y = 40 ksi)'
f'_c = 2000 psi	$M/bd^2 = 100 + 25{,}000p$
f'_c = 3000 psi	$M/bd^2 = 100 + 28{,}000p$
f'_c = 4000 psi	$M/bd^2 = 100 + 30{,}000p$
f'_c = 5000 psi	$M/bd^2 = 100 + 31{,}000p$
For high tensile reinforcement	(f_y = 65 ksi)
f'_c = 2000 psi	$M/bd^2 = 100 + 45{,}000p$
f'_c = 3000 psi	$M/bd^2 = 100 + 48{,}000p$
f'_c = 4000 psi	$M/bd^2 = 100 + 50{,}000p$
f'_c = 5000 psi	$M/bd^2 = 100 + 51{,}000p$

The above equations do not necessarily apply when p is less than one percent.

For over-reinforced beams, it will be assumed that failure occurs at a definite limiting strain in the concrete. This is the basis of the method proposed by Jensen.[3] Using this assumption, the following equations can be deduced: (see Fig. 1)

Total force in steel = $f_s\, pbd$

Total force in concrete = $f'_c\, bx$

Substituting $f_s = E_s \left(\frac{d-x}{x}\right) \epsilon_c$

and $f'_c = E'_c\, \epsilon_c$

we get $$E'_c\, \epsilon_c\, bx = E_s\, \epsilon_c \left(\frac{d-x}{x}\right) pbd \quad \ldots\ldots\ldots\ldots (1)$$

therefore $$\frac{x}{d} = \frac{-n'p}{2} + \frac{\sqrt{(n'p)^2 + 4n'p}}{2} \quad \ldots\ldots\ldots\ldots (2)$$

where $n' = \frac{E_s}{E'_c}$.

also $$M = f'_c\, bx \left(d - \frac{x}{2}\right)$$

$$\frac{M}{bd^2} = f'_c \frac{x}{d} \left(1 - \frac{x}{2d}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

It will be noticed that E'_c is defined as the ratio of the ultimate strength of the concrete to the assumed limiting value of strain in the concrete. In other words it is the secant modulus at failure. Consequently the term n' in the above equations is comparable in some ways to the modular ratio used in conventional theory; its value depends upon the strength of the concrete and it will be shown later that it can be determined

approximately from the results of tests. If E_s is assumed to be constant then the value of n' will depend only upon the strength of the concrete.

The critical percentage can be determined from Eq. (1) and (3) but the resulting equation is unwieldly. It is simpler to solve for the critical percentage graphically, or to use an empirical equation as proposed later in this paper.

TEST SPECIMENS AND PROCEDURE

For economy and for ease of experimentation small specimens 2 in. x 3 in. x 4 ft were used. To reproduce actual working conditions as closely as possible, they were loaded by dead weights in a single lever testing machine built for the purpose (Fig. 2).

Two grades of steel having yield points of about 40 and 65 ksi, and four grades of concrete with strengths ranging from 2000 to 5000 psi were used in the tests. Crushed limestone ranging in size from ½ in. to No. 4 was used as coarse aggregate and sand with a fineness modulus of about 2.2 was used as fine aggregate. The ratio of fine to coarse aggregate was 1 to 1.5 (by absolute volumes), this ratio being chosen so as to obtain a mix that could be placed easily. Concrete strengths were determined from 4 x 8 in. cylinders. Steel ratios varied from 0.5 percent to 4.7 percent. Plain bars were used throughout these tests and in some cases it was necessary to use cold rolled steel. Experience showed that this material was unsatisfactory unless its surface was roughened by rusting and consequently this was done. Details of the specimens are given in Table 1.

During each test, measurements were made of the strain on the upper surface of the concrete and on the lower surface of the steel using Huggenberger extensometers. A record was also kept of deflections.

The stress-strain curve for the reinforcement was determined separately for each beam using a Templin stress-strain recorder.

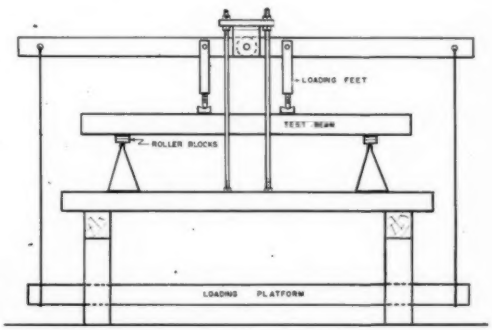

Fig. 2—Sketch of testing machine showing beam in position

TABLE 1—PARTICULARS OF THE BEAMS AND THEIR ULTIMATE MOMENTS OF RESISTANCE

No.	Reinf. diam., in.	No. of bars	b, in.	d, in.	p, percent	f'_c, psi	f_s, ksi	Ult. B.M., ft-lb	$\frac{M}{bd^2}$ psi	$\frac{M}{bd^2}$ *corrected, psi	Cause of failure
Beams with Structural Grade Reinforcement											
4201	¼	1	1.99	2.47	1.18	1780	36.6	434	438	479	Yield of steel
4202	¼	1	2.00	2.43	1.56	1570	39.7	542	548	552	Yield of steel
4203	⅜	1	2.00	2.40	2.30	1930	44.4	852	753	678	Yield of steel
4204	⅜	1	2.00	2.43	3.05	1970	43.9	897	912	835	Uncertain, prob. steel
4205	½	1	2.04	2.45	4.00	1970	39.2	900	885	900	Crushing concrete
4206	⅜	2	2.03	2.49	4.75	1930	44.4	937	895	928	Crushing concrete
4301	¼	1	2.03	2.62	1.10	2070	36.6	494	426	466	Yield of steel
4302	¼	1	2.02	2.53	1.50	2760	39.7	600	558	565	Yield of steel
4303	⅜	1	2.00	2.48	2.30	2770	42.8	897	879	820	Yield of steel
4304	¼	2	2.01	2.42	2.38	2750	39.4	834	848	862	Yield of steel
4305	⅜	1	1.98	2.48	3.05	2770	44.8	1074	1058	945	Yield of steel
4306	⅜	2	2.10	2.40	2.98	2750	40.7	972	965	950	Yield of steel
4307	½	1	2.04	2.45	3.98	2770	40.8	1254	1220	1200	Uncertain, prob. steel
4308	⅜	2	2.02	2.47	4.54	3330	42.8	1242	1210	1100	Crushing concrete
4401	¼	1	2.00	2.54	1.14	4500	38.6	520	480	498	Yield of steel
4402	⅜	1	2.02	2.57	1.45	4500	39.7	609	545	548	Yield of steel
4403	⅜	1	2.00	2.51	2.31	3950	43.5	954	905	830	Yield of steel
4404	¼	2	2.04	2.42	2.35	4500	36.6	750	750	820	Yield of steel
4405	⅜	1	2.01	2.48	3.00	3950	44.0	1189	1160	1050	Yield of steel
4406	⅜	2	2.00	2.41	3.16	4500	39.7	1044	1082	1090	Yield of steel
4407	½	1	2.07	2.63	3.67	4170	40.8	1520	1271	1220	Crushing concrete
4408	⅜	2	2.02	2.44	4.71	4490	43.4	1567	1552	1390	Crushing concrete
4501	¼	1	2.00	2.44	1.19	5180	38.6	486	492	510	Yield of steel
4502	⅜	1	2.02	2.42	1.57	5180	39.7	572	586	592	Yield of steel
4503	⅜	1	2.00	2.45	2.32	5180	43.5	954	954	876	Yield of steel
4504	¼	2	2.00	2.36	2.46	5120	36.6	777	833	910	Yield of steel
4505	⅜	1	2.00	2.50	2.96	4960	44.0	1184	1136	1030	Yield of steel
4506	⅜	2	1.99	2.40	3.18	5180	37.4	1029	1081	1160	Yield of steel
4507	½	1	2.05	2.60	3.74	4450	41.5	1514	1320	1270	Yield of steel
4508	⅜	2	2.04	2.42	4.66	5170	43.5	1594	1594	1534	Yield of steel
Beams with High Tensile Reinforcement											
6201	⅜	1	2.03	2.48	0.53	1780	58.3	314	302	336	Yield of steel
6202	¼	1	2.03	2.51	0.97	2080	86.0	726	681	515	Yield of steel
6203	⅜	1	2.01	2.69	1.40	2150	88.0	872	745	692	Uncertain, prob. concrete
6204	⅜	1	2.03	2.69	2.00	2150	75.8	872	716	662	Crushing concrete
6205	⅜	1	2.06	2.46	2.25	1950	75.8	844	810	836	Crushing concrete
6206	⅜	1	2.04	2.58	2.84	2080	73.5	942	832	802	Crushing concrete
6207	½	1	2.02	2.50	3.85	1915	75.2	787	748	770	Crushing concrete
6208	½	1	2.02	2.45	3.91	2120	75.2	870	862	822	Crushing concrete
6301	⅜	1	2.01	2.42	0.55	2750	62.5	314	317	330	Yield of steel
6302	¼	1	2.00	2.65	0.93	3200	83.5	812	696	542	Yield of steel
6303	⅜	1	2.00	2.58	1.47	3200	72.0	992	890	810	Uncertain
6304	⅜	1	2.00	2.45	2.33	2760	75.8	957	957	1040	Crushing concrete
6305	⅜	1	2.01	2.60	2.86	3200	74.0	1226	1082	1015	Crushing concrete
6306	½	1	2.00	2.47	3.94	2760	75.2	1009	990	1080	Crushing concrete
6401	⅜	1	2.00	2.43	0.55	4490	64.0	344	348	353	Yield of steel
6402	¼	1	2.04	2.67	0.90	4140	82.0	762	627	500	Yield of steel
6403	⅜	1	2.00	2.61	1.51	4140	81.5	1257	1110	885	Yield of steel
6404	⅜	1	2.10	2.40	2.26	4490	75.8	1189	1180	1050	Crushing concrete
6405	⅜	1	2.06	2.58	2.80	4140	74.0	1396	1212	1172	Crushing concrete
6406	½	1	2.01	2.47	3.90	4190	75.2	1362	1330	1270	Crushing concrete
6407	½	1	2.00	2.47	4.08	4190	62.1	1452	1430	1370	Crushing concrete
6501	¼	1	1.99	2.68	0.92	4450	86.0	872	740	562	Yield of steel
6502	⅜	1	2.03	2.58	1.44	4450	88.0	1322	1178	870	Yield of steel
6503	⅜	1	1.98	2.45	2.30	4770	65.0	1357	1365	1365	Yield of steel
6504	⅜	1	2.01	2.43	2.33	4870	75.8	1197	1205	1235	Crushing concrete
6505	½	1	1.99	2.64	3.71	4450	65.0	1542	1390	1500	Crushing concrete
6506	⅜	2	2.01	2.48	4.58	5450	75.8	1645	1590	1460	Crushing concrete

*Corrected to f_y = 40 or 65 ksi for beams failing due to yield of steel, and to the nominal concrete strength for beams failing due to crushing of concrete.

MANNER OF FAILURE

All the beams failed as a result of excessive flexural strain, sufficient shear reinforcement being provided, where necessary, to prevent diagonal tension failures. In a few instances, bond failures occurred and these beams were repeated with the shear reinforcement welded to the longitudinal reinforcement. This proved effective in preventing bond failures.

There is some doubt as to the proper criterion of failure in a reinforced concrete beam. Obviously, a beam has failed if it can no longer sustain the maximum load that has been placed upon it. But concrete is a plastic material, especially when highly stressed, and a beam may fail under a somewhat smaller sustained load than that which would cause failure if applied for a very short time. The ordinary test indicates neither the ability of a beam to support an instantaneous load nor its ability to support a sustained load. A brief series of tests indicated that loads about 5 percent less than those reported in the regular tests could be sustained indefinitely, and that loads about 5 percent greater could be sustained momentarily. In the present series of tests the beams were loaded with increasing loads until failure was considered to have occurred, each test taking about 15 minutes.

It should, perhaps, be noted that beams reinforced with high tensile steel deflect considerably before complete failure occurs. If a limiting deflection of span/120 had been established as a criterion of failure, some of the results would have been substantially lower than those reported.

PRESENTATION OF RESULTS

The results of all tests on the 57 beams are summarized in Table 1 and presented graphically in Fig. 3a, b, c and d. It will be noticed that the ultimate moments have been corrected for variations in the quality of the steel or the concrete. When failure was due to yield of steel, the moments were adjusted to the standard values assumed for the two grades of steel, namely, 40 ksi for structural grade steel, and 65 ksi for high tensile steel. When failure occurred primarily because of crushing of the concrete, values were corrected to allow for deviation of the concrete strength from the intended figure. When the cause of failure was uncertain, both adjustments were tried, and the one that appeared to give the better results was used.

Beam failure due to yield of steel

There is a close agreement between the actual failing load of beams in which the steel yields, and the loads calculated according to theory. Either the conventional theory or the plastic theory is satisfactory for

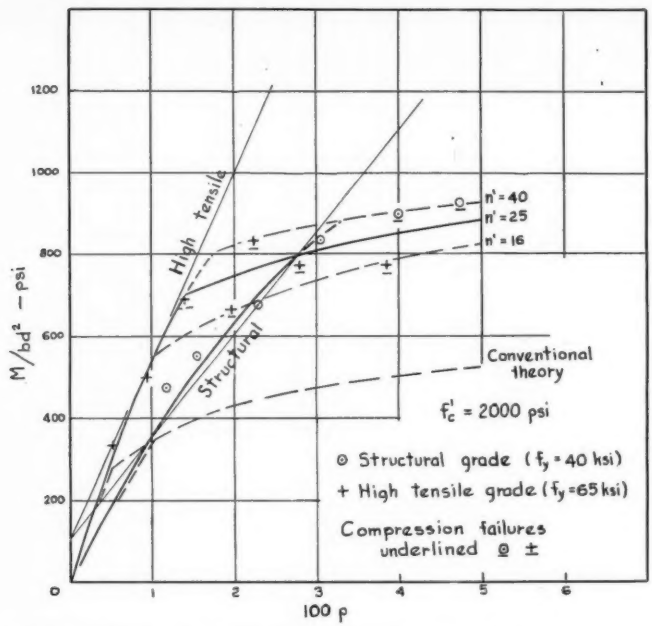

Fig. 3(a)—Comparison between test results and theoretical curves of M/bd^2, f'_c = 2000 psi

such beams, but concrete stresses should be ignored if the conventional theory is used. The results agree more closely with the plastic theory but the difference is not important. It was noticed in carrying out these tests that bars having good bond characteristics gave better results than bars that were smooth or bright.

Beam failure due to crushing of concrete

In the case of over-reinforced beams the results show considerable scatter. This is to be expected because it is not possible to determine accurately the properties of the concrete in the beam; also, the strength of such beams appears to be influenced by the nature of the cracks on the tensile side of the beam, and these in turn are dependent on the bond characteristics of the reinforcement and its stress-strain curve. With the materials used, it proved to be impossible, even with the most careful experimental work, to reduce the scatter of the results below the limits shown in Fig. 3a, b, c and d. It seems probable that the variation would have been much greater if the experiments had included beams of widely different dimensions.

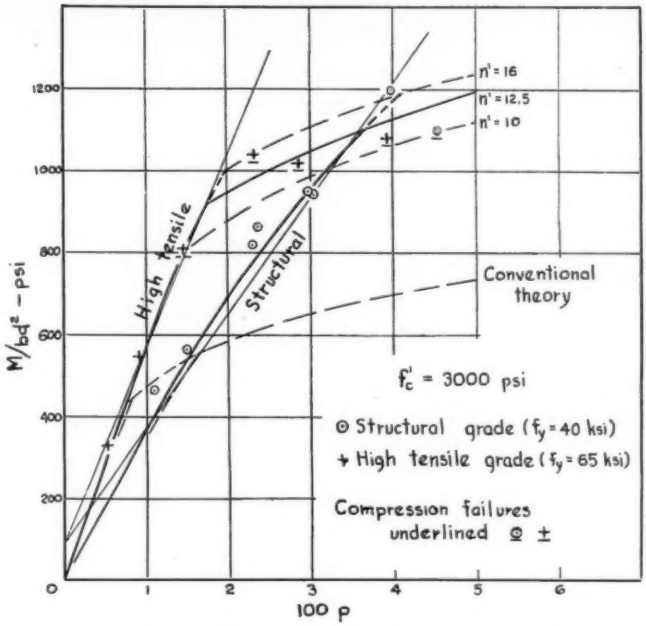

Fig. 3(b)—Comparison between test results and theoretical curves of M/bd^2, f'_c = 3000 psi

Since the strength of over-reinforced beams can not be predicted accurately, it follows that the critical percentage can not be determined accurately. On the other hand, it is evident that the plastic theory represents the behavior of over-reinforced beams in a satisfactory manner, and it is simply a matter of selecting the appropriate value of n' for any particular strength of concrete.

The values of n' for the various grades of concrete are shown graphically in Fig. 4, and the critical percentages corresponding to these values are shown in Fig. 5 and 6. In the latter, lines have been drawn indicating the critical percentage according to various theories. It will be noted that there is fairly close agreement between the observed upper limit values and those predicted by Jensen. The agreement is less satisfactory with the Whitney or Cox formulas, and it is doubtful if the critical percentage can be satisfactorily expressed by an equation of the form $p = k\dfrac{f'_c}{f_y}$.

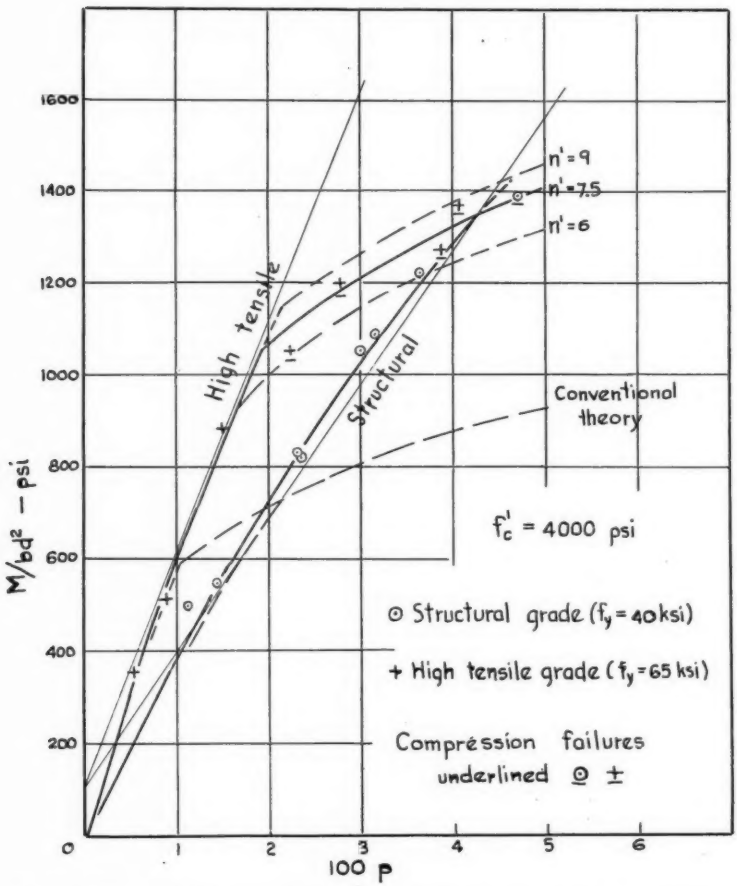

Fig. 3(c)—Comparison between test results and theoretical curves of M/bd^2, f'_c = 4000 psi

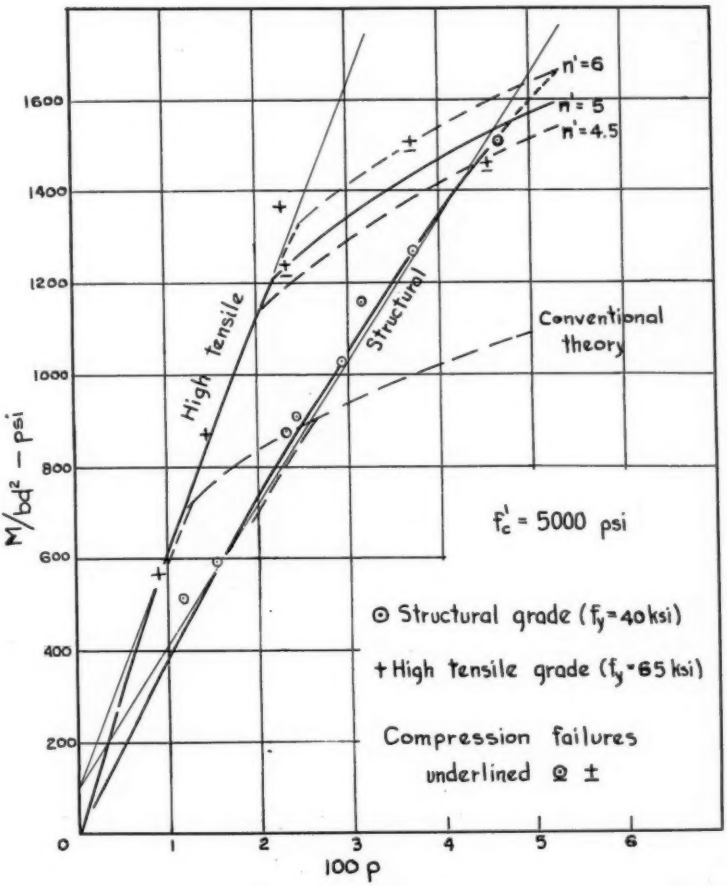

Fig. 3(d)—Comparison between test results and theoretical curves of M/bd^2, f'_c = 5000 psi

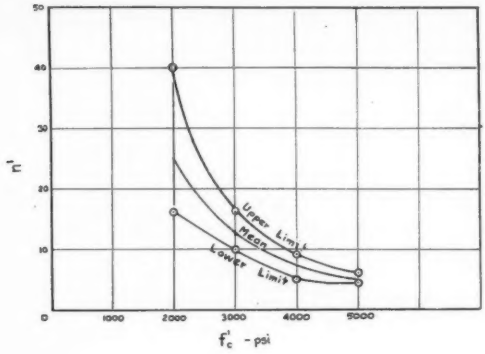

Fig 4—Values of n' deduced from tests

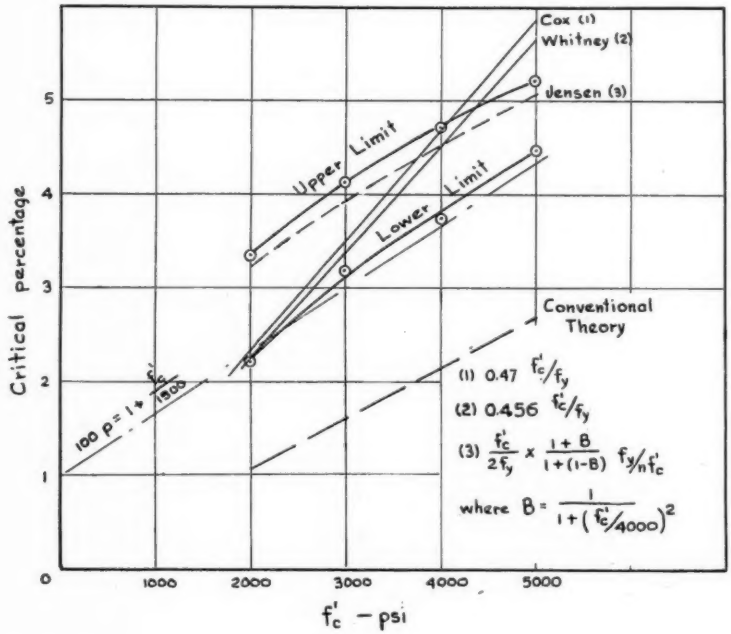

Fig. 5—Critical percentage of reinforcement using structural grade steel (f_y = 40 ksi)

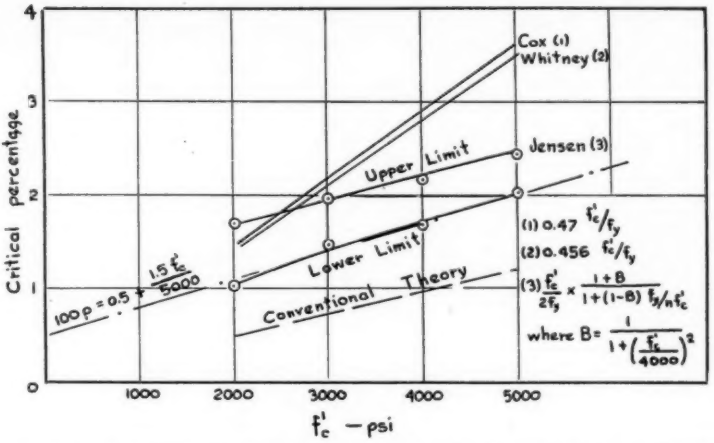

Fig. 6—Critical percentage of reinforcement using high tensile grade steel (f_y = 65 ksi)

Although the results quoted by Jensen give support to the upper limit values obtained in these experiments, some beams failed at considerably smaller loads and it would therefore seem prudent to regard the limiting percentage as the lower limit values. These range from 2.2 to 4.2 percent for structural grade steel, and from 1 to 1.9 percent for high tensile steel, depending on the strength of the concrete. It is probably sufficiently close if we express the critical percentage by the following formulas:

For structural grade reinforcement $$100\,p = 1 + \frac{f'_c}{1500}$$

For high tensile reinforcement $$100\,p = 0.5 + \frac{1.5\,f'_c}{5000}$$

CONCLUSIONS

1. The ultimate strength of the beams agreed fairly closely with results calculated according to the conventional theory provided that, (a) failure was due primarily to yield of steel and (b) high concrete stresses were ignored.

2. The ultimate strength of the beams tested agreed fairly closely with results calculated according to the plastic theory in all cases, whether failure was due primarily to yield of steel or to crushing of concrete.

3. The critical or balanced percentage of reinforcement was much greater than the conventional theory indicated. Its value could not be determined very closely, but a safe lower limit appeared to be given by the following formulas:

For structural grade reinforcement $100\ p = 1 + \frac{f'_c}{1500}$

For high tensile reinforcement $100\ p = 0.5 + \frac{1.5\ f'_c}{5000}$

4. Small specimens are satisfactory for many experimental purposes. They are easy to make and handle and facilitate testing by dead weight loadings.

ACKNOWLEDGMENT

The authors are indebted to J. Burgoin of the Royal Military College, Kingston, Ont., for assistance in the laboratory; to the Canada Cement Co., Ltd., for cement, and to the Steel Co. of Canada, Ltd., and the Burlington Steel Co., Ltd., for reinforcing steel.

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Discussion of a paper by S. D. Lash and J. W. Brison:

The Ultimate Strength of Reinforced Concrete Beams*

By BOYD G. ANDERSON and EDWARD COHEN, FRANK A. BLAKEY, J. W. GRAHAM, Jr., HOMER M. HADLEY and AUTHORS

By BOYD G. ANDERSON and EDWARD COHEN†

The authors are to be commended for adding more test data to the existing store of information regarding the ultimate strength of concrete beams. It is, however, unfortunate that they have not correlated their results with the many derived from previous investigations. This is particularly true since the materials and dimensions of the specimens used by the authors are far from being representative of practical construction.

An earlier and excellent paper by R. H. Evans[1] compared the plastic theories of L. J. Mensch, H. Kempton Dyson, K. Hajnal-Konyi, V. P. Jensen and C. S. Whitney. Evans discussed the stress-strain relationship of various size and strength members under flexural loads and compared the ultimate load computed by the various theories with the actual results obtained from forty-one 5 x 10 x 100-in. test beams having concrete strengths ranging from 975 to 7250 psi and steel ratios from 0.0047 to 0.067.

In several earlier papers Whitney[2] has shown by reference to 309 beam tests that, for concrete strengths between 3000 and 6000 psi, the ultimate resisting moment for reinforced concrete beams is approximately equal to $0.333bd^2f'_c$ if the beam is over-reinforced and primary failure occurs by crushing of the concrete. According to Whitney, the critical percentage of steel reinforcement necessary to develop this full compressive strength of the concrete is $p_o = 0.456f'_c/f_y$.

The writers have taken the liberty, in Fig. A and B, of presenting data from the tests by Evans[1] and of replotting the authors' test data to the same, and what are possibly more informative, parameters which separate the concrete strength as a variable. Fig. A shows a plot of the ultimate value of $M/bd^2f'_c$ versus f'_c based on the test results presented by the authors and by Evans for all beams having more than the critical percentage of steel.

*ACI JOURNAL, Feb. 1950, *Proc.* V. 46, p. 457. Disc. 46-29 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings*, V. 46.

†Ammann and Whitney, Consulting Engineers, New York, N. Y.

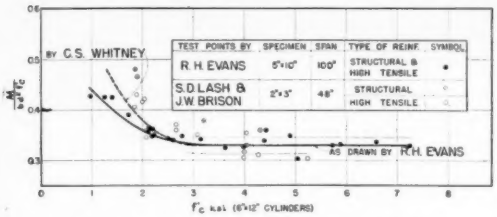

Fig. A—Comparison of $M/bd^2f'_c$ versus f'_c values

Evans' curve also shows a constant value of $M/bd^2f'_c = 0.333$ for concrete strengths in excess of 3000 psi and deviates from that originally presented by Whitney only for values of f'_c below 2500 psi. The data presented by the authors also gives a reasonably good correlation with the same curve, and even closer agreement with the curve by Whitney for concrete strengths below 2500 psi.

If the data are plotted on the basis of the critical percentages presented by the authors, a considerable scatter occurs, with many points falling below 0.333. This may indicate that the authors have misinterpreted tension failures which in the later stages apparently failed by crushing of the concrete.

In this connection, it must be remembered that a slight stretching of the steel can reduce the concrete compression area and cause, in an under-reinforced beam, what would appear to be a compression failure. Moreover, such misinterpretation would explain the authors' inability to correlate their test results with the value of the critical steel percentage as given by $p_o = 0.456\, f'_c/f_y$.

When however, those specimens having steel percentages smaller than $0.456\, f'_c/f_y$ are eliminated from the plot the correlation is greatly improved and the results appear as shown in Fig. A. For concrete strengths below about 2500 psi the large concrete strains which take place as the ultimate moment is approached cause a considerable drop in the position of the neutral axis and consequently a sharp rise in the ratio $M/bd^2f'_c$.

With present methods and materials it is not probable that structural concrete will have a strength less than 2500 or 3000 psi, and if a weaker concrete is used, a greater factor of safety is appropriate. Therefore, it appears that a value of 0.333 for $M/bd^2f'_c$ can be used for concrete of any strength whenever the steel percentage is greater than the critical.

Since the test results for concrete strengths below 2000 psi are erratic (Fig. A) and since they have only limited application to current design practice, it seems reasonable to proceed on the basis of the higher strength beams, knowing that the results for those with lower strengths will then err only on the side of safety.

Fig. B has been plotted using all points derived from beams having concrete strengths greater than 2000 psi. The curve is drawn as derived by Whitney on the basis of tests by Humphrey and Losse, the U. S. Geological Survey, Columbia University, and Slater and Lyse. The tests by Evans lend further evidence of its accuracy and there is good agreement with the authors' data. Their statement that "the results show considerable scatter" in the case of over-reinforced beams appears justified only for the lower concrete strengths. For the higher strengths, the data is in relatively good agreement with that of numerous other investigators if the critical steel percentage is computed on the basis of Whitney's formula.

It may be of interest to note that an error of approximately $\frac{1}{16}$ in. in the depth of the reinforcing steel of the authors' test beams will result in material discrepancies between the actual and theoretical moment capacities. The possible range of error due to this consideration alone is shown on Fig. A and B by the dotted lines either side of the theoretical curve.

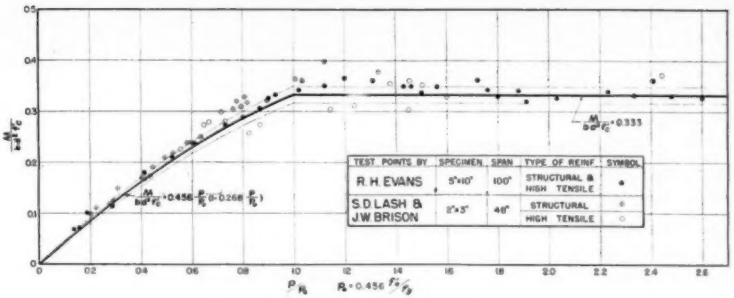

Fig. B—Comparison of $M/bd^2f'_c$ versus p/p_o

The authors' assumption of a uniform concrete stress equal to f'_c at failure is not in accordance with the views of Whitney, the tests by Evans, or with test results in general. Evans has added further verification to the drop-off in stress at maximum strain by an independent determination of stress-strain relations for various concrete mixes. He states that "there is no doubt that the stress-strain curves for the usual concrete mixes in practice do bend over and that the resulting stress distribution in beams near the failing loads must be modified considerably." On the basis of strain measurements on actual test beams Evans has also corroborated the proposition that the line of action of the actual compressive forces correspond closely to that given by an equivalent rectangular stress block of intensity $0.85 f'_c$.

In presenting their material, the authors have apparently neglected the variation of crushing strength with the dimensions of the test cylinder. If we presume that a 6 x 12-in. cylinder is standard, then the 4 x 8-in. cylinder strengths used for the authors' control tests should be reduced 4 percent below the values of f'_c presented in the paper. (This correction has already been made for Fig. A and B.)

The authors mention briefly that a series of tests carried out by them indicates that variations in the speed of testing can produce a variation of only plus or minus 5 percent in the load carrying capacity of reinforced concrete beams. Although the writers agree that small changes in the rate of loading will not appreciably affect the test results, they wish to point out that there is the possibility of much greater variation in moment capacity than is implied in the above statement. High speed cylinder tests show strength increases up to 15 percent[3] and indicate that more rapid rates of loading than have been used thus far would yield even greater increases. In addition, tests recently made at Massachusetts Institute of Technology[4] indicate that the moment capacity of reinforced concrete beams may increase as much as 35 percent when subjected to high rates of loading. Although this particular data may not concern most static design problems it is of considerable importance for rational proportioning of members in structures subjected to earthquake or other shock loadings and for the investigation of

existing structures for the same conditions. Furthermore, long time tests[5,6,7] indicate that the permanent loading capacity of concrete cylinders may be as much as 15 percent less than the load carried at the conventional testing rate. It has already been pointed out[5] that the available data on plastic flow under sustained loads may indicate a need for the re-evaluation of the present methods of expressing the factor of safety for members carrying permanent loadings. It would appear, therefore, that the variations in the rate of loadings can result in variations in strength of at least 50 percent rather than 5 percent as suggested by the authors.

The authors are unreasonably pessimistic in describing the scatter of their test points, particularly their speculation that greater variation might have been obtained if the test beams had been of widely different dimensions. The beams used by Evans were approximately eight times as large as the authors', while the beams used by Whitney to derive the theoretical curves of Fig. A and B were of numerous sizes and height to width ratios.

In converting the test values of M/bd^2 to standard steel strengths of 40 ksi and 65 ksi and to nominal concrete strengths it appears that the authors have used a method which is subject to error. Since there will be a change in the depth of the concrete stress block and therefore in the lever arm of the resisting forces, a change in f_y does not cause a proportional change in M/bd^2. Where the ultimate moment capacity of a specimen which failed by yielding of the steel is revised to a substantially higher value, there is the possibility that this moment may actually correspond to a concrete failure. If it seems desirable to work with standard values of f_y, the conversion may be made consistent with the actual test results by multiplying the actual percentage of reinforcement by the ratio of the actual yield strength to the assumed yield strength. This gives a corrected steel area which supplies the same total tension at the standard strength as the actual area gives at the actual strength.

For the case of a concrete failure the value of M/bd^2 may be adjusted by the authors' procedure. However, it is also necessary to adjust the value of p by the same ratio, unless one accepts the fact that for steel ratios greater than p_o, M/bd^2 remains constant. The authors have thus inadvertently accepted the accuracy of the curve drawn in Fig. B of this discussion. The authors also appear to be inconsistent in their computation of n'. The values of n' which the authors have computed are based on the test values of $M/bd^2f'_c$ and p. In the curves of Fig. 4 of their paper, these points have been plotted at abscissas corresponding to the nominal concrete strengths. Since n' is a function of f'_c this procedure is obviously in error. At least in this case it would have been preferable to work with the actual test values.

It appears to the writers that it would be simpler to work with the actual test values of M/bd^2 as is done in this discussion and eliminate the effect of concrete strength by dividing by the actual f'_c.

Generally the authors' tests and comments are interesting and informative. However, the writers believe that the best use for this and other data is the

examination and evaluation of existing theories. When sufficient statistical data is available to show the need, certain modifications of the original empirical coefficients may be necessary. The authors' data, however, seems to be an added confirmation of the original factors rather than evidence for a change.

The writers have made their comparisons on the basis of Whitney's presentation in particular because his method represents a simple and practical design method not only for beams such as those discussed by the authors, but also for the more important region of members with and without compression steel and members subject to combined bending and direct stress. The plastic theory method of design, if confined to simple flexure, offers only the advantage of permitting the use of higher percentages of reinforcement and the more accurate determination of ultimate strength. However, the most important advantage in the use of the plastic theory is that it provides a method which yields realistic values for the strength of members under combinded bending and direct stress. Also of great value is the availability of a single, rapid and logical method for the design of most types of reinforced concrete members under most types of loading. The formulas proposed by the authors, however, are based on a limited series of flexural tests and are necessarily limited to the condition of simple bending.

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By FRANK A. BLAKEY*

As one of the many who have covered the same field as the authors, the writer is interested in their use of small, model-size beams. At the University of Tasmania under the direction of Prof. A. Burn, the writer tested a series of 8 x 5 x 99-in. reinforced beams; later, at the University of Cambridge in England, another similar series of 2 x 1.5 x 24-in. beams. The general outcome of this work is in agreement with the conclusions of the authors, but it is worth emphasizing that the ultimate strength theory applied equally well to the models as to the full size beams.

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The authors are reticent on two points which may be important in this matter. The first is the amount of shear reinforcement used and the second is the manner of failure of the concrete. The writer found it impossible to obtain compressive failure in the concrete unless heavy shear reinforcement was used to strengthen the end section outside the section subject to constant moment. No stirrups were used in the central section, but this has not been the case for other workers in this field. The writer believes that stirrups may have a binding effect on the concrete in the compression zone, increasing the strength of this region in much the same way as spiral reinforcing increases column strength.

The manner of failure of over-reinforced beams is of interest. In models tested by the writer in which the ratio of the width of the beam to its effective depth was about one, crushing was localized near one of the load points, and after the shattered material was removed the failure resembled that of a cube subject to compression. However, for the full size beams tested at the University of Tasmania where the corresponding ratio was nearer 1.5, there was no evidence of crushing, but the over-reinforced beams failed in the compression zone when the concrete split on a line between the two loading points which seemed to follow the lines of maximum shearing stress indicated in photoelastic experiments.

It would be of great interest to know whether compressive failures were possible in beams as built in practice. The writer has examined over 200 published test results of which few refer to other than rectangular beams. The restraint to which the compressive zone of a rectangular beam loaded at one or two points is subject, is much less than that experienced in the corresponding zone of a T-beam, subject to uniform loading. The extent of the effects mentioned in this and the preceding paragraphs and the fundamental reasons for such differences in behavior will only be satisfactorily established when the fundamental theory of rupture of concrete is developed.

The writer has examined Jensen's method of determining the critical percentage of steel in reference to all the published results mentioned earlier, and finds that it is in agreement with the trend apparent from these results, but that it is not in good agreement with the individual results. An alternative method, which may become satisfactory when more information is available has been developed by Professor Burn and the writer.

Since the plane sections remain plane, $k/1 - k = e_c/e_s = r$. Let $x = pfy/f'_c = k$ according to the ultimate strength theory. Therefore, $x/1 - x = r$ and $x = r/1 + r$. When the steel percentage p, and thus x, reaches its critical value ($x = x_{cr}$), the yield strain in the steel and the ultimate strain for the concrete will be reached simultaneously.

Let $r' = e'_c/e_y$, where e'_c is the ultimate strain for the concrete and e_y the yield strain of the steel. Thus, $x_{cr} = r'/1 + r'$.

This may be made to fit the available data as well as Jensen's formula, by a suitable choice of e_c'. The value of e_y is easily determined and does not vary widely. The value of e_c' will probably be no more variable than any

other property of concrete. This value would have to be obtained from bending tests since the ultimate strain under such conditions is greater than that occurring in an ordinary compression test.

The writer would be interested to know whether the authors have tested any beams reinforced in compression as well as tension. The writer has made some such tests but the results are not in agreement with Whitney's theory or, in fact, any other theory at present available; in particular, anomolous results have been obtained for the shearing strength of such beams.

By J. W. GRAHAM, Jr.*

Most reinforced concrete structures are continuous, and thus must satisfy both statics and continuity. If a plastic theory is to replace conventional theory, it must be able to predict the ultimate strength of a *structure*—not just the ultimate strength of simple beams (which need satisfy statics alone).

Design moments are calculated by the elastic theory, which assumes that moments are directly proportional to angle changes. An elastic frame or arch analysis can be combined with relaxation techniques to predict the ultimate strength of a concrete structure, but only if $\Delta\phi/\Delta M$ can be predicted for all parts of the structure and for all ranges of concrete stress. The writer is unable to see how the theory presented in this paper could be extended to such an analysis; it is based on a relationship of stress and strain (Fig. 1) which implies a total stress-strain diagram that is not in agreement with experimental evidence. The stress-strain curve for medium strength (2000-4000 psi) concretes in compression is approximately a parabola with the vertex, f'_c, at a strain of about 2000 millionths. Assuming an equivalent rectangular distribution of stress at one section seems only to cloud the issue; it may be consistent with statics but it is not consistent with continuity.

By HOMER M. HADLEY†

The basic objection to the conventional straight-line beam theory which leads the authors of this paper and the other cited writers to propose various substitute theories for concrete in flexure is this: when beams fail by flexural compression in the concrete, the fiber stress computed by the conventional theory is far in excess of the compressive strength of the same concrete measured in a standard 6 x 12-in. cylinder. This discrepancy is regarded as a flagrant weakness in the conventional theory and has produced a number of substitute theories which have this common characteristic: under them flexural failure occurs at a computed fiber stress just equal to the 6 x 12-in. cylinder strength. Such agreement is regarded as a validation of the substitute theory. In Fig. 1 the authors present the pattern of stress distribution which they have employed and it will be noted that it is at f'_c, the 6 x 12-in. cylinder strength, that failure occurs.

But why should there be this agreement of flexural and compressive strength? What is there about a 6 x 12-in. cylinder to make its strength the precise measure of beam strength? The types of loading are wholly different. The cylinder

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"fibers" seek to bow laterally to avoid load and do bulge outward at mid-height. The compressive "fibers" of a beam are on its concave side and must buckle against that curvature. The entire cross-sectional area of the cylinder is subjected to uniform pressure. The beam pressures, despite all the contrary assumptions such as that exemplified in Fig. 1, are not uniform because the outermost fibers have a greater lever arm about the tension steel than those at the neutral axis and, in the economy of nature, they automatically have not only a greater deformation but likewise a greater stress.

All of the substitute theories are presumed to have a physical basis for their stress patterns. Whitney's is based on the observations of two seniors at the University of Wisconsin on stress-strain relationships of cylinders beyond the ultimate. Jensen's is a curious combination of actual stress-strain measurements of cylinders in the "elastic range," *i.e.*, up to the ultimate, with the speculations of Saliger as to what *may* happen beyond the ultimate, the combination being then modified by a process of "idealization" and "approximation." The authors of the present paper apparently find their justification in Saliger although not stating what specific evidence Saliger has presented.

Now the fact is, as the writer has pointed out elsewhere,* these patterns of stress-distribution do not agree with concrete. A cylinder in the last stages of "plasticity," shattered, fissured and structurally worthless can still sustain a total load of 5000 lb by friction and aggregate interlock only. Fig. C presents a stress-strain record of such a cylinder. Observe, as a matter deserving careful attention and thought, that the stress-strain curve gives no automatic indication of the cracked and fissured condition of the cylinder. Anywhere in its long descent from the ultimate strength it attained at the uppermost reaches of its straight-line elastic range—maximum load 39,500 lb—a sudden increment of load would have caused it to fly into countless fragments. Companion cylinders so treated did precisely that, even though showing no outward visible cracks. This is what happens in Whitney's "plastic range." As for the Jensen combination of reality with the Saliger speculation nothing remotely resembling that could be found in the test of a dozen cylinders. Nor can anyone else find any resemblance to it in any other similar cylinder tests. It is pure "make-believe." In short, the situation is this: concrete does behave elastically up to the ultimate; beyond that point it is structurally worthless and any theory or any pattern of stress-distribution which conjectures anything to the contrary automatically falls in the same category.

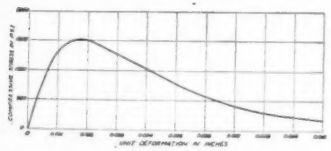

Fig. C—Stress-strain diagram for 6 x 12-in. cylinder

Returning to the 6 x 12-in. cylinder, let the question be repeated: what is there about it to make its compressive strength the precise measure of flexural strength? True, the 6 x 12-in. cylinder is a convenient test specimen and from long habit and custom it has become the sole and only basis on which strengths of concrete are

*Civil Engineering, Apr. 1950, p. 29 and correction May 1950, p. 49; ACI JOURNAL, Nov. 1950, *Proc.* V. 47, p. 257.

compared in America. But what of that? Put its concrete in another 6-in. cylinder of a different height, and a different strength is measured. In still another 6-in. cylinder of a still different height, yet another strength is measured. (See Gonnerman, *Proceedings*, ASTM, 1925, Part 2, p. 237.) As a matter of fact, perfect agreement of flexural and compressive strength, if it did occur, would be by the sheerest, most fantastic of coincidences. So why be at such pains to fabricate a theory—"plastic," "ultimate," "ne plus ultra" —whereby flexural beam stress is computed to be the 6 x 12-in. cylinder strength? All this ado is particularly amusing when no valid physical basis to support the refined theories can be produced—only conjecture, speculation, thin air.

Slater and Lyse,* found beam strengths computed by straight-line formula to be about 1.75 times the 6 x 12-in. cylinder strength with 2000-lb concrete; about 1.50 times the cylinder strength with 3000-lb concrete; about 1.40 times the cylinder strength with 6000-lb concrete. Now if engineers will really assimilate these ratios and forget the great delusion that a beam ought to fail at a computed fiber stress just equal to the 6 x 12-in. cylinder strength, they can know the load carrying capacity of a structure, can know when failure is to be anticipated, can have concrete working stresses set at values to give reasonable, rather than silly, factors of safety and can still preserve and retain the beautifully simple straight-line theory which everyone knows and which incidentally has reality to support it. Why develop an elaborate if flawless mathematical theory as a substitute for the straight-line theory when no physical justification for the substitution can be produced?

AUTHORS' CLOSURE

Several members have discussed the shape of the stress-strain curve for concrete and have questioned the soundness of the stress distribution assumed in the Saliger and Whitney analysis. The authors were careful not to make any definite statement concerning the stress-strain curve for concrete. They assumed that whatever this might be, the stress distribution in a beam at failure could be represented with sufficient accuracy by a rectangular block. The correlation between test results and calculated values show that this assumption is not unreasonable. Nevertheless, the authors do not agree with Hadley's statement that concrete does behave elastically up to the ultimate. This certainly is not true unless the effects of plastic flow are ignored and such effects cannot be ignored except perhaps for impact loads and other loads of short duration. In this connection the comment of Anderson and Cohen are pertinent. For static loadings, and these are the only type of loadings the authors considered, there seems little doubt that appreciable redistribution of stresses occurs as a result of plastic flow.

Graham is concerned about the effects of a nonlinear stress-strain relationship upon the ultimate strength of a structure. This is beyond the scope

*Slater, Willis A. and Lyse, Inge, "Compressive Strength of Concrete in Flexure as Determined from Tests of Reinforced Beams," ACI JOURNAL, June 1930, *Proc.* V. 26, p. 831.

of the present paper, but it has been considered by many writers and the problems do not appear insuperable. References may be made to papers by Glanville and Thomas,[1] Van den Broek,[2] J. F. Baker,[3] A. L. L. Baker[4] and M. R. Horne.[5] Even though the stress-strain curve for concrete itself is not linear, the deflections (and hence the angle changes), up to the useful maximum load, are a linear function and of the same order as would be predicted by the elastic theory. Hence it appears that a structure may be analyzed by the same methods that apply to elastic structures and then the individual members designed to conform with the theory of plasticity. Although this method is not entirely inconsistent, the errors introduced are small compared to the usual approximations used in structural engineering. In any case it seems reasonably certain that the ultimate strength of the structure must depend upon the ultimate strength of its component parts.

The principal point at issue is this—can the test results be explained satisfactorily on the basis of the conventional theory? The answer is no—the critical steel ratio is much higher than the conventional theory would indicate and the strength of over-reinforced beams much greater. Hadley maintains, if the authors understand him correctly, that the reason for this difference is that the wrong criterion is used to determine the strength of the concrete. Now it is quite possible to multiply the cylinder strength by a factor and in this way arrive at a better agreement. Unfortunately, as pointed out by Hadley, the factor that is used has to depend upon the strength of the mix. It might be worthwhile to bolster up a worn out theory by introducing one empirical factor but it seems foolish to introduce two such factors.

An alternative approach is to discard the cylinder as a measure of the compressive strength of concrete and substitute something else, for example, an over-reinforced beam. This has been the practice in some European countries. Such information as has come to the attention of the authors indicated that the result of such tests are not greatly different from the results of cylinder tests, *i.e.*, there is a more or less constant relationship between the two. So much of our basic knowledge of the properties of concrete is exposed in terms of cylinder strengths that it seems doubtful if the test should be discarded.

Anderson and Cohen criticize the use of an f'_c value based on tests of 4x8-in. cylinders. The latter were used partly as a matter of convenience and partly because they were in better proportion to the size of the test specimens.

Apologies are due to Evans, as well as to others, for apparently ignoring the excellent work that they have done in this field. The authors made no claim to originality and they felt it best not to try and interpret other investigators' results. They are still of this opinion in spite of the careful study given by Anderson and Cohen to the test results published by Evans and also to those in the authors' paper. The value of Whitney's work in this field is beyond all question but nothing is well established about the relationship $M = 0.333 f'_c bd^2$ except that 0.333 is equal to one third. Of the 309 tests referred to by Anderson and Cohen, the great majority were beams failing

TABLE A—COMPARISON OF OBSERVED AND CALCULATED ULTIMATE MOMENTS

Method of calculation	Calculated moment / Observed moment	Mean square error
Test results reported by Lash and Brison (17 beams)		
Whitney*	1.027	±0.158
Lash and Brison†	1.016	±0.051
Conventional (modified)‡	1.000	±0.062
Test results reported by Evans (23 beams)		
Whitney*	0.944	±0.095
Lash and Brison†	0.984	±0.093
Conventional (modified)‡	1.000	±0.148

* $M = 0.333 f'_c bd^2$

† $M = f_c' bd^2 k \left(1 - \frac{k}{2}\right)$ where $k = \frac{-n'p}{2} + \sqrt{\frac{(n'p)^2 + 4n'p}{2}}$

n' from Fig. 4.

‡ $M = C_o \frac{f'_c}{2} jkbd^2$ where $k = -np + \sqrt{(np)^2 + 2np}$

and $n = \frac{30{,}000}{f'_c}$, $j = 1 - \frac{k}{3}$

$C_o = 1.570$ Lash and Brison tests.
$C_o = 1.462$ Evans tests.

by tension of the steel not by crushing of the concrete. The evidence in support of Whitney's formula for over-reinforced beams is quite inconclusive.

A comparison has been made between the test results for beams failing due to crushing of concrete and the calculated values according to (a) Whitney, (b) the formula proposed by the authors, and (c) the formula $M = C_o(f'_c/2)jkbd^2$, *i.e.*, the conventional formula modified by a coefficient C_o. This coefficient has been chosen so as to make the average ratio of calculated moment to observed moment equal to unity. These calculations have been made both for the beams reported in the authors' paper, and for those in Evans' report.

From Table A it is clear that the method proposed by the authors gives better results than the Whitney method and that both methods are superior to the conventional method. In the latter case it will be noted that there is a significant difference in the values of C_o in the two series of tests.

The authors do not prefer the method of presenting test results in which $M/f'_c bd^2$ is plotted against f'_c. It is true that this shows a tendency for $M/f'_c bd^2$ to approach 0.33 especially for values of f'_c between 3000 and 4000 psi. On the other hand the scatter is considerable and there is no way of judging which of a number of factors are responsible for the scatter. The same graph could be used almost equally well to support quite different theories as to ultimate strength. Moreover, such agreement as is indicated has been obtained by omitting beams with less than what Anderson, Cohen and Whitney regard as the critical percentage of steel. The authors are not inclined to accept the suggestion that they have misinterpreted tension failures as compression failures. Those cases in which doubt existed at the time of test were recorded as uncertain and in the remainder there was little or no doubt as to the primary cause of failure. Warnings of impending failure due

to steel yield were indicated by the readings of the extensometer attached to the reinforcement.

The authors recognized that small errors in determining the dimensions of the beams would have serious effects and such measurements were made with more than usual care. It is unlikely that the errors exceed $\frac{1}{100}$ in.

In discussing the significance of the rectangular stress block of intensity $0.85 f'_c$ which has been used by Whitney and others, Anderson and Cohen entirely overlook the other dimensions of this stress block. Whitney selected a value of $0.537 d$ for the depth of this block on the basis of a limited series of test results. He could equally well have assumed an intensity of f'_c instead of $0.85 f'_c$ and arrived at a slightly different figure for the depth of this block. Why introduce an empirical factor when none is required?

The authors were glad to hear from Blakey that ultimate strength theories apply equally well to full size beams and to small specimens. The authors' experience with full size test beams has been confined to studies made by students, but the results have been satisfactory within the usual range of experimental error.

Blakey inquires about shear reinforcement. This consisted of a continuous spiral of No. 13 gage soft iron wire having a pitch of about an inch and extending from the loading points to the supports. There was no shear reinforcement in the central portion of the beam.

The method proposed by Blakey and Burns for determining the critical steel ratio appears both logical and simple. It requires assumptions as to the limiting strains in both steel and concrete. If the former is regarded as the yield point strain, it is then only necessary to determine a value for the latter. One set of such values can be obtained from the values of n' given in the paper. It can easily be shown that on the basis of Blakey's approach the critical steel ratio is given by

$$p_{cr} = \frac{f'_c}{f_y}\left[\frac{1}{1+\frac{E_s e'_s}{n' f'_c}}\right] \quad \text{and if } e'_s = \frac{f_y}{E_s} \qquad p_{cr} = \frac{f'_c}{f_y}\left[\frac{1}{1+\frac{f_y}{n' f'_c}}\right]$$

This equation gives results that agree reasonably well with those presented.

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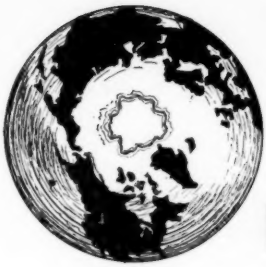

Letters from Readers*

BY WAY OF SYNOPSIS

KATHERINE MATHER and RUTH D. TERZAGHI discuss the leaching of lime from concrete and the resultant strength loss.

ROBERTO FORLANI describes recent work in Italy with no-fines concrete.

J. J. POLIVKA describes concrete barges built in Europe.

RUDOLPH RAUCH cites the need for information about expansion joints in buildings.

HENRY COMACK adds further comments on the problem of making corrections for free moisture in aggregates.

Leaching of Lime from Concrete (LR 46-48)

KATHARINE MATHER, Jackson, Miss., discusses the relation of the structure of cement paste to the removal of lime.

Readers of the JOURNAL who have studied the paper by Ruth D. Terzaghi, "Concrete Deterioration in a Shipway,"[1] will be interested to consult her subsequent paper "Concrete Deterioration Due to Carbonic Acid."[2] These two papers present significant data and hypotheses concerning the role of carbonic acid in concrete deterioration. In her more recent paper (p. 147) she states: "The most conspicuous chemical effects of carbonic acid attack on concrete are the removal of lime from the paste and the conversion of a part or of all the remaining lime to calcium carbonate. . . . Experiments carried out by Tremper[3] have shown that loss of strength is proportional to loss of lime; loss of strength becomes complete when the concrete has lost about one-half of its original lime content. A similar relation between loss of strength and loss of lime was found in a group of concrete specimens taken from a shipway.1"

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The relation between strength loss and lime loss noted above suggests two possible explanations:

(1) That the removal of crystalline calcium hydroxide ($Ca(OH)_2$) from the locations it previously occupied in the paste is enough in itself to reduce the strength by the amounts found.

(2) That the loss of $Ca(OH)_2$ is significant only as an indication that the structure of the hydrated paste has been altered by the passage of the solutions that removed the $Ca(OH)_2$.

The first assumption appears untenable if applied to concrete more than a few months old, since pure dicalcium silicate (C_2S) hydrates without the formation of $Ca(OH)_2$, and develops strengths which are ultimately comparable with those of hydrated tricalcium silicate (C_3S) as shown by Brown and Carlson.[4]

The second assumption may not be sufficiently defined, since we know so little of the structure of the hydrated paste. But to speculate:

(1) If we follow Powers and Brownyard[5] (pp. 971-992), the paste consists principally of "amorphous" solid material bonded together but containing gel pores, capillary pores, and crystals of $Ca(OH)_2$ and probably calcium sulfoaluminate. The size of the solid units of gel is somewhat smaller than 140 Å (possibly about 30 Å-)[6]; the mean diameter of the gel pores is from 20 to 40 Å. The capillary pores are very much larger than the gel pores (Powers and Brownyard). The permeability of dense paste is about equal to that of granite, but the permeability of concrete is much greater (underside voids, bleeding channels, etc.) The permeability of well cured concrete with water-cement ratios above about 0.5 is largely determined by the by-passes around the gel and the by-passes around the paste in the structure as a whole. Initial absorption in a dry paste specimen takes place almost exclusively in the capillaries outside the gel.

(2) Now if we consider Brown and Carlson[4], calcium hydroxide crystallizes at numerous centers in a mixture of cement and water, while gel develops around cement grains and tends to grow into available free space. In thin sections of nondeteriorated concrete, there is usually a tendency for $Ca(OH)_2$ to border aggregate particles or project from them, but some of the hydroxide crystals sit apparently isolated and surrounded by gel. This discussion neglects the third dimension but it still seems reasonable that some of the hydroxide should crystallize within areas of gel.

(3) To put together parts of (1) and (2), may we not assume for the purpose of argument that the crystals of hydroxide that we see under the microscope in nondeteriorated concrete are located principally in the "by-passes around the gel and the by-passes around the paste"? Or to put it in another way, may we not assume that the crystals of hydroxide are relatively easy of access to solutions or vapors which may remove or alter them without material chemical or physical disruption of the gel?

(4) However, there is more than 60 percent CaO in modern cement and $Ca(OH)_2$ usually amounts to 10-20 percent of the area of paste in thin sections of concrete several years old, which leaves the largest part of the CaO dispersed through the gel, probably as adsorbed Ca and O ions. (Brown and Carlson remark on the ease of carbonation of porous gel produced by the hydration of coarse cement, say carbonation progresses more rapidly in areas of gel than in areas of hydroxide.)

(5) To put together parts of (1), (3) and (4), may it not be assumed that the loss of lime which is associated with loss of strength is in large part the loss of Ca (and O?) which was adsorbed on the gel, and which could not be reached without changes (loosening?) of the state of aggregation of the gel?

The discussion set forth above is a preliminary attempt to relate observed behavior of concrete, a chemical mechanism of attack, and data on the microstructure and composition of the paste. It is suggested that the results reported[1,3] and the means of attack hypothecated[1,2] (aggressive carbon dioxide) require participation by both the lime present in hardened concrete as crystalline $Ca(OH)_2$ and also that present in ionized form in the amorphous gel. This suggestion is perhaps also derivable from the data given in Table 3, p. 996, of Terzaghi's paper.[1]

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RUTH D. TERZAGHI, Winchester, Mass., contributes further comments on the relation between lime loss and strength loss of concrete.

The available data regarding the relationship of lime loss to strength loss of concrete are unfortunately rather meager. They do not provide an adequate basis for final conclusions regarding the nature of those processes associated with lime loss, which lead to loss of strength. Nevertheless, it is extremely useful to examine the data critically, as Mrs. Mather has done. The present writer welcomes the opportunity to join the discussion of this interesting and important topic.

As Mrs. Mather has indicated, a part of the lime leached from concrete is derived from crystalline calcium hydroxide and a part from the gel phase. Loss of lime from either phase probably contributes to loss of strength, and the writer sees no immediate possibility of evaluating the relative importance of these two sources of leached lime. With regard to the first, it can hardly be doubted that the removal of any solid constituent, such as calcium hydroxide, from the paste would reduce the strength of the concrete. With regard to the second source of leached lime, *i.e.*, the gel, experience shows that loss of strength of concrete subject to leaching continues after all of the crystalline calcium hydroxide has disappeared. Hence a large measure of the strength loss must be

associated with leaching of lime from the gel. Thus the two explanations of the relation between strength loss and lime loss which Mrs. Mather suggests at the beginning of her discussion refer to complementary processes and should not be regarded as competing hypotheses.

In attempting to interpret data regarding the relation between loss of lime and loss of strength, it should be remembered that strength is presumably lost as soon as lime is removed either from the crystalline or from the gel phase, irrespective of whether the lime is redeposited as a carbonate elsewhere in the concrete, or completely removed from it. Under some circumstances, it is likely that only a small percentage of the leached lime is redeposited; under such conditions, data showing loss of lime from the concrete represent with fair approximation the quantity of lime lost from the original constituents of the paste. Under other circumstances, much of the leached lime is redeposited as calcium carbonate. In this case the quantity of lime removed from the concrete as a whole represents only a part of the total leached from the original constituents of the paste. In this connection it is important to note that some of the published observations refer only to that part of the lime which is lost from the concrete, and do not provide any indication as to how much was leached from the paste but retained in the concrete as calcium carbonate.

In view of the relations discussed in the preceding paragraph it is not surprising that there is considerable variation in the ratio of strength loss to lime loss. The two published sets of data for concrete exposed to aggressive carbonic acid bearing water (Tremper,[1] 1931 and Terzaghi,[2] 1948) show a relatively high ratio of strength loss to lime loss (roughly 2 percent of the original strength lost for each 1 percent of the original lime content lost). On the other hand, data given by Ruettgers, Vidal and Wing[3] (1935) for concrete exposed to nonaggressive water indicate a much lower ratio (roughly 0.6 percent loss of strength for each 1 percent of original lime lost). The high ratio indicated by the data for concrete exposed to aggressive carbonic acid bearing water is perhaps to be explained as follows. A large part of the lime leached from the gel was redeposited in the concrete as calcium carbonate. Analyses given in Table 3 of the writer's 1948 publication[2] indicate that the total quantity of lime reprecipitated as calcium carbonate was as large as or even larger than the quantity removed from the concrete. The migration of lime from the gel phase to a calcium carbonate phase contributes conspicuously to a decrease in strength, but the quantity of lime which thus migrates is not represented in the total lime lost from the concrete. Hence the ratio of loss of strength to loss of lime from the concrete as a whole is high. In contrast, nonaggressive water, containing only a moderate quantity of free carbonic acid, may, perhaps, remove lime in solution as

calcium hydroxide and reprecipitate very little lime as calcium carbonate. If this explanation is correct, the quantity of lime removed from concrete by such water should be nearly as great as that removed from the paste; hence the ratio of loss of strength to loss of lime would be low.

According to the hypothesis just presented, loss of strength is a function of the total quantity of lime leached either from crystalline calcium hydroxide or from the gel phase, regardless of whether it is removed from the concrete or redeposited. This quantity is equal to the sum of the lime lost from the concrete plus the quantity of lime reprecipitated in the form of calcium carbonate. An adequate test of this hypothesis could be made only on the basis of chemical analyses* and results of compression tests on a large number of leached specimens. Should such data fail to show the relationships predicted by the hypothesis, then it will become necessary to search for causes of strength loss other than simple removal of lime from crystals of calcium hydroxide and from the gel phase. Among such additional causes which may require consideration is the disruption of the gel due to local volume changes associated with crystallization of calcium carbonate.

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No-Fines Concrete (LR 46-49)

ROBERTO FORLANI, della Hammersan University, Naples, Italy, describes the work on no-fines concrete.

No-fines concrete is a concrete composed of cement and coarse aggregate, the sand being omitted so as to leave many uniformly distributed voids throughout the mass. The concrete may be used for load bearing external and party walls, partitions and chimney breasts.

Strength

It has been found that the compressive strength of no-fines concrete of the mixes given in Table 1 is sufficient for two or three story domestic

*The chemical analyses must provide all the information required for computing the quantity of lime lost from the concrete as well as the quantity retained in the concrete by redisposition. Loss of lime from the leached concrete may be determined, as explained in the appendix to the writer's 1948 publication, by comparing the ratio of lime to one of the relatively insoluble oxides (silica, alumina, or alumina-plus-iron) for the original cement to the same ratio for the altered paste. The quantity of lime redeposited as $CaCO_3$ is equivalent to the quantity of CO_2 present in the altered paste.

TABLE 1—COMPRESSIVE STRENGTH OF NO-FINES CONCRETE

Aggregate	Cement-aggregate ratio	W/C ratio	Strength, psi at 3 months	Compressive strength of axially loaded 8 ft x 18 ft x 18 in. piers, tons per sq ft
¾ to ⅜ in. gravel	1-10	0.45	860	33
Light aggregate	1-6	0.60	410	16
Brickwork	Common bricks and mortar		1-3 cement	50-70

wall construction of the usual thickness for solid brickwork. Table 1 gives existing data on the strength of mixes.

Water resistance

A no-fines concrete wall with suitable construction and good detailing is comparable to an 11-in. cavity brick wall as regards resistance to moisture penetration. Owing to the cellular structure, capillary attraction virtually does not occur in no-fines concrete, and water penetration is limited to about twice the diameter of the coarse aggregate, *i.e.*, 1½ in. Thus, an external wall of rendered no-fines concrete consists of a wall with no capillary suction, faced with a rendering with some capillary action, and there is no tendency for water to be drawn into the wall from the rendering.

Weight

The weight of no-fines concrete is about two-thirds that of normal concrete. The hydrostatic pressure on formwork during placing is considerably less than with normal concrete.

Thermal insulation

Experience has shown that 8 in. rendered and plastered external walls of a no-fines concrete using a heavy aggregate, are not troubled with condensation. The test results so far available indicate that the insulation of such a wall is similar to that of a brick wall of the same thickness, rendered and plastered.

Labor requirements

Unskilled workmen can be easily and quickly trained in correct procedures for mixing and placing no-fines concrete, although it is important that those supervising the training understand the essential difference between no-fines and normal concrete.

Cost

The cost of a rendered no-fines concrete wall 8 in. thick is about the same as an 11-in. cavity fair-faced brickwork, depending largely on the efficient reuse of formwork and the organization of the site.

Formwork

General requirements for formwork are: to ensure speed of erection and removal by unskilled labor and to ensure that while an efficient system of standardization is used, it does not necessarily limit design or erection.

This type of construction has been developed to a high degree of efficiency. By the polarization of the concrete through low tension electricity which was induced in the formwork to permit the removal of the forms in a matter of hours instead of days, the building process has been speeded up to an even greater degree.

Thin Wall Concrete Ship Construction (LR 46-50)

J. J. POLIVKA, consulting engineer, Berkeley, Calif., describes concrete barges built in Europe.

I read with interest Mr. Mac Leay's article in the November issue of the JOURNAL describing the successful use of ¾-in. and 1½-in. concrete walls. I remember the difficulties we encountered back in 1921 when we built the first reinforced concrete Danube barge with walls only 1¾ in. thick. These concrete barges are still in use in Yugoslavia. They were built by the Syndicate for Construction of Concrete Ships and Freight Cars in Prague and Vienna; Victor Lazarus, Walter Stross and the writer were the consultants and designers. Further details were published in *Beton u. Eisen*, 1921, No. 6, pp. 76 and 77, and No. 14/15, p. 173.

Expansion Joints (LR 46-51)

RUDOLPH RAUCH, Engineering Dept., Virginia Pulp and Paper Co., New York, emphasizes the need for more information on expansion joints in buildings.

Expansion joints in buildings should be given more consideration. An inspection of buildings with and without contraction joints for the occurrence or absence of cracks might yield a lot of information which could be put to good use by designers of future buildings. Such an inspection would be quite an undertaking but if ACI Members gave their experiences with either type of structure, the task would be a lot easier.

Making Corrections for Free Moisture (LR 45-19)*

HENRY COMACK, civil engineer, Flushing, N. Y., offers a rebuttal in the discussion started in the May JOURNAL.

This is written in response to the questions and comments received concerning my previous statement in this column published in the May 1949 JOURNAL, particularly by Sam Davidson, Edward E. Bauer and T. M. Kelly in the September 1949 JOURNAL. The following should serve to clarify the data submitted.

The proposed method adjusts the designed surface-dry batch weights of aggregate, and is based upon the percentage of free moisture in the sample of *damp* aggregate. The latter can mean only one thing. I heartily agree that if a percentage is obtained based upon dry weight, the correct value is obtained by multiplication. But, the percentage based upon dry weight can only be considered and defined as a correction factor and not the true percentage of free moisture in a specimen. This is why the subject has been brought up at the known risk of stirring up a hornets' nest; as a matter of fact, this question has frequently been asked: How can dry material have a percentage of moisture? A further illustration of the scientific implication is as follows:

A sample of material such as wet clay etc., weighs 1000 g before drying. After drying it weights 500 g. By the standard method, the moisture content computed is 100 percent. Actually, the true moisture content is 50 percent. At 100 percent where is the residue?

It is the writer's opinion that the standard method could be modified so that the percentage of moisture be a significant value, and to be applied by division as indicated in the formula. This, I believe would eliminate much confusion on this subject. In answer to Mr. Kelly, dry weight refers to saturated surface dry material.

*See also ACI JOURNAL, May 1949, *Proc.* V. 45, p. 684 and Sept. 1949, *Proc.* V. 46, p. 65.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Prismoidal thin-slab structures—II

A. J. Ashdown, *Concrete and Constructional Engineering* (London), V. 44, No. 10, Oct. 1949, pp. 317-322 — Reviewed by Glenn Murphy

This installment includes a discussion of unsymmetrical prismoidal structures, with vertical or sloping ends. Formulas are developed for stresses at critical sections.

Comparative tests on two reinforced concrete beams

Giuseppe Rinaldi, *Giornale del Genio Civile* (Rome), V. 87, No. 3, Mar. 1949, p. 109 and No. 7-8, July-Aug. 1949, p. 385 — Reviewed by Gennaro Mianulli

The author tested to destruction two beams of equal dimensions and quality of concrete, reinforced with wires of high elastic limit. In one beam the wire was pre-stressed, in the other the wires were sunk without pre-tension.

Details of test data are described. The results are tabulated individually and used to compare the strength of the two beams at different stages of testing.

Reinforced concrete

A. L. L. Baker, Concrete Publications Ltd. (London), Sept. 1949, 360 pp. Price 15s — Reviewed by F. H. Hall

The author has attempted to cover the theory and practice of reinforced concrete from the fundamental concepts to advanced analysis of indeterminate structures. The presentation is made from a technical standpoint, which combined with the rapid coverage, requires the reader to have already a solid understanding of the subject matter to be able to follow the author.

On the design of an eccentric pressure on a group of reinforced concrete columns placed at the vertices of a regular polygon

Luigi Massei, *Giornale del Genio Civile* (Rome), V. 87, No. 5, May 1949, p. 245 — Reviewed by Gennaro Mianulli

This paper deals with the development of an approximate method to compute eccentric loads on a group of columns placed at the vertices of a regular polygon. These columns, straight or inclined, are generally found supporting cylindrical tanks and spheres.

Very simple formulas are presented, which with the aid of numerical tables permit rapid computation. Tables have been compiled for a group of 4, 6, 8, 10 and 12 columns.

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Change of stress in reinforced concrete columns and other construction due to plasticity and shrinkage (Die Spannungsumlagerung infolge Dauerplastizitat und Schwinden in Stahlbetonstutzen und anderen Tragwerken)

H. CRAEMER, *Beton- u. Stahlbetonbau*, V. 43, 1944, No. 1/2 — Reviewed by RUDOLPH FISCHL

Formulas are derived to determine the stresses in unloaded reinforced concrete structures such as columns and arches, due to shrinkage and plastic flow, as a function of time.

New theater in Piombino (Il nuovo cineteatro di Piombino)

GIORGIO NEUMANN, *L'Industria italiana del cemento*, Apr. 1949 — Reviewed by J. J. POLIVKA

The stone masonry building designed by Prof. G. Forti, architect, and by the author as structural engineer, has roof and floors of reinforced concrete. Details of the extremely economical structure are described and illustrated.

Requirements for construction materials (Norme per l'Accettazione e per il collaudo dei materioli da costruzione)

Consiglio Nazionale delle Ricerche, Rome, 1940 — Reviewed by GENNARO MIANULLI

These regulations cover the use of reinforced and plain concrete, mortar, brick and natural stone, tile and paving materials. They are issued by the Italian National Board of Research, and provide detailed specifications covering work in which concrete and reinforced concrete are to be used.

The booklet has chapters on definitions, quality of cement and aggregates, proportions and mixing of concrete, reinforcement and descriptions of testing methods.

Bibliography (Documentation bibliographique)

Bulletin de Documentation No. 2, Centre D'Etudes et de Recherches de L'Industrie des Liants Hydrauliques (Paris), 1948-49, 115 pp.

Major subjects for which references are given are raw materials, cement, lime, plaster, tests and analyses of lime and cement, concrete and mortar, and precast and prestressed construction. A second section gives more detailed summaries of the more important articles listed. While many of the references are to American publications a substantial number of articles in European engineering periodicals are mentioned.

The effect of electrolysis on reinforced concrete

Concrete and Constructional Engineering (London), V. 44, No. 7, July 1949, pp. 228-229 — Reviewed by GLENN MURPHY

This article is an abstract of two recent reports published by the British Electrical and Allied Industries Assn. The electrical resistance increases with time, more rapidly for DC than for AC and more rapidly when the concrete remains wet. If alkaline salts are present they inhibit the deposition of calcium carbonate near the cathode and the increase in resistance does not occur. Bond strength may be greatly affected by electrolytic treatment, being either increased or almost completely destroyed, depending on current density and alkalinity.

Nomograms for members subjected to combined stress

J. S. SAVONA, *Concrete and Constructional Engineering* (London), V. 44, No. 7, July 1949, pp. 219-221 — Reviewed by GLENN MURPHY

This article is a continuation of one published in Feb. 1948, and presents two nomograms for members subjected to nonaxial loading. The first nomogram is for a symmetrically reinforced rectangular section which develops compressive stress only, while

the second is for a similar section developing both tensile and compressive stresses. Ordinary theory is used in developing the nomograms and bending is assumed to occur about an axis parallel to one of the faces of the section.

Buildings at Filton for the British Overseas Airways Corporation

Concrete and Constructional Engineering (London),
V. 44, No. 7, July 1949, pp. 209-215

Reviewed by GLENN MURPHY

The reinforced concrete construction described includes a thin slab prismatic roof, precast columns and beams in three wings of the building, and the cast-in-situ floor slabs and staircase. The design of the V-shaped roof members, which are continuous over five spans of 30 ft each is outlined and construction details are presented. The structure contains 95 precast columns of three different types. Average 28-day concrete strengths were 3750 and 4750 psi for columns and beams respectively.

The design of prestressed reinforced concrete sections (Die Berechnung von Stahlbetonquerschnitten mit Vorspannung)

L. PISTOR, *Betonstein Zeitung* (Wiesbaden),
No. 6, June 1949, pp. 93-98

Reviewed by RUDOLPH FISCHL

This paper deals only with members where the reinforcement is prestressed before concreting and where the prestresses will be transmitted to the hardened concrete by means of continuous anchorage such as bond, friction or by special mediums.

A careful theoretical treatment and a scheme for practical application is given, the criterion for single or double reinforcement is discussed and general as well as special cases are studied.

Glassy state and properties of cement (Glaszustand und Eigenschaften der Zemente)

ELSNER VON GRONOW, Vandenhoeck & Ruprecht (Göttingen),
1948, 52 pp.

Reviewed by RUDOLPH FISCHL

This booklet contains a scholarly study based on the statement that the physical chemistry deals mainly with the crystal structure of matter, but has an insufficient knowledge of the glassy structure.

Portland cement contains about 30 percent glassy substances, hydraulic cement about 90 percent. Uncertainties in the early strength, quantity of mixing water, heat development during hardening, etc., are dependent on the vitrified substances in cement.

Different methods and tests to determine the amount of glassy substances in cement are discussed and the dependency of physical and chemical properties of the vitrified particles is investigated.

Water-cement ratio versus cement-water ratio

N. M. PLUM, *Beton og Jernbeton* (Copenhagen),
V. 1, No. 2, Sept. 1949, pp. 48-60

Reviewed by EIVIND HOGNESTAD

This paper, which was also published in *Ingenioren* (Copenhagen), Sept. 10, 1949, pp. 742-747, raises the question of whether the water-cement or the cement-water ratio is most convenient in practice. In an effort to gather more information regarding this problem, the author sent a note to 26 concrete specialists all over the world and requested statements of opinion. The English version of the note is quoted in full, and the essential parts of the 18 answers received are quoted in their original languages. These answers are 11 in favor of the C/W ratio, and 2 in favor of the W/C ratio. Five answers do not express a clear choice. On the basis of this poll, the author strongly recommends the future use of the cement-water ratio in Denmark. The article carries an excellent international bibliography.

Determination of economical dimensions of a reinforced concrete T-beam subjected to bending moment and shear

LUIGI MONTERISI, *Giornale del Genio Civile* (Rome), V. 87, No. 3, Mar. 1949, p. 119

Reviewed by GENNARO MIANULLI

The author discusses the assumptions advanced by various authorities in establishing an economical cost equation for a reinforced concrete T-beam. To Berger's assumptions are added a few considerations in order to determine the quantity of steel required for the bending moment and shear.

A cost equation including the quantity of reinforcement is established and by successive derivations a new formula is arrived at, which fixes the most economical depth for the web. The formula is applicable to cases of fixed and variable depth of the web.

Continuous arches on elastically yielding piers. Theoretical computation compared with results of experimental analysis.

V. KOLOUSEK AND J. BLAZEK, *Technicky Obzor*, Journal of Czechoslovak Society of Engineers SIA (Prague), V. 57, No. 2, Feb. 10, 1949, pp. 21-28

Reviewed by J. J. POLIVKA

If continuous arches are supported by piers resting on elastically yielding soil the theoretical analysis of internal forces becomes very intricate and cumbersome. It is shown that a relatively quick solution may be obtained by tests on small-size models in a way similar to the deformeter method of the late Prof. George E. Beggs of Princeton University and that the results corroborate the theoretical analysis. A thorough analysis is presented including influence lines of bending moments, direct stress and shear. A simplified method is proposed which yields satisfactory results as compared with both theoretical and experimental analyses.

Elastic rigidity of continuous structural members

KAREL VALEK, *Technicky Obzor*, Journal of Czechoslovak Society of Engineers SIA (Prague), V. 57, No. 3, Mar. 10, 1949, pp. 41-48

Reviewed by J. J. POLIVKA

The author refers to his previous paper published in *Technicky Obzor*, 1944, pp. 101-109, and investigates structural members of various stiffness and having constant and variable moments of inertia (haunched beams and columns). Tables were calculated for continuous beams of equal symmetrical spans having three typical distances of conjugate points from the adjacent supports, u or $v = 0.25$, 0.33 and 0.40. Similar tables were calculated for continuous beams with symmetrical spans, however with increasing stiffness and with variable stiffness. As further examples, continuous beams on elastic supports, central hexagonal section of a grain elevator and a central rectangular unit of a structural network are discussed.

The reconstruction of New Holland pier

Concrete and Constructional Engineering (London), V. 44, No. 10, Oct. 1949, pp. 309-315

Reviewed by GLENN MURPHY

The London and North Eastern Railway Co. has recently completed the reconstruction of the ferry landing at New Holland on the Humber. The 21-ft ramp is over 220 ft long and is to serve for passengers, vehicles and cattle. The structure is supported on 76 reinforced concrete piles 15 in. square and from 45 to 49 ft long. A rapid hardening portland cement and a 1:1½:3 concrete was used for the columns, braces, beams and slabs, while precast prestressed concrete planks 4 in. thick were used for the floor of the structure. A vertical floor load of 200 psf and a lateral force equivalent to a 600-ton boat berthing at 1 knot was assumed for the design. Design and construction details are described and a number of excellent photographs are included.

Elastic-plastic theory of bending in reinforced concrete beams (Theorie elasto-plastique de la flexion dans les poutres en beton arme)

CHAMBAUD, *Annales de L'Institut Technique du Batiment et des Travaux Publics*, No. 101 Béton, Béton Armé No. 10, Nov. 1949

The paper reports the completion of investigations into the elastic-plastic deformations occurring when reinforced concrete members are loaded to failure in bending.

The following points are of special interest: (1) a theoretical discussion leading up to a new form of the stress-strain diagram of concrete, which gives a simple illustration of the state of quasi-neutral equilibrium existing near the point of rupture, (2) examples of the use of theory given in graphical form, so that the relations between the different factors involved can be readily understood.

Introduction of prestressed concrete in Italy (Sull' introduzione del precompresso in Italia)

G. NEUMANN, *Il Cemento, Rivista tecnica della costruzione* (Milan), V. 7, July 7, 1949 Reviewed by J. J. POLIVKA

The author reviews the history of prestressing mentioning Koenen (1907), Charles Rabut, Abeles, and Jackson of San Francisco who had the idea of prestressing 20 years ahead of Rabut. Practical applications were introduced by Hoyer in 1938 and Freyssinet in 1939. Thermo-electric method of prestressing concrete was introduced by the Czechoslovak Ruml in 1930. In Italy the work of Freyssinet and Magnel-Blaton is mostly appreciated. Italy has plenty of excellent cements but little steel so that economy of reinforcement has always been of great importance. The author has proposed methods of saving steel since 1910, some of them being adopted later in Germany (*Beton u. Eisen*, 1941 and 1942, *Beton u. Stahlbeton*, 1943).

Reinforced concrete structures (Il Cemento Armato)

LUIGI SANTARELLA, Ulrico Hoepli (Milan), 1940 Reviewed by GENNARO MIANULLI

This text book for engineering schools and office, well known in Italy, was first published in 1927 and has been brought to date and enlarged in the 11th edition by Mario Baroni and Maria Miattazzi Sanatarella, daughter of the author.

The original Volume I has been divided into two, and chapters dealing with the latest developments in concrete technologies, torsion, shrinkage and prestressed concrete have been added to complete the present first volume on theory of design.

The second volume is entirely new and treats extensively foundations, flat slabs, rigid frames, arches and silos.

The text book, limited to reinforced concrete buildings, has an unusually large number of problems, tables and diagrams.

Insulation of concrete with bitumen (Die Isolierung von Beton mittels Bitumen)

H. VIERHELLER, *Zement-Kalk-Gips*, V. 2, No. 8. Aug. 1949, p. 155 Reviewed by RUDOLPH FISCHL

Two kinds of bitumen are generally used for insulation: a solution in organic mediums such as benzol or others, or an emulsion with water.

In many cases failures occurred in tests of the water impermeability of bituminous coated slabs. It was found that a solution in benzol was used throughout for these coatings. As both bitumen and benzol are water repellent, even a very small amount of surface humidity in the concrete will prevent adherence of the coating to the concrete. But an emulsion will work very well on a wet surface.

The author suggests the use of a bitumen emulsion first and after successive drying two more coats of bitumen solutions in benzol. The results were very satisfactory and the insulation withstood the pressure test in every respect.

Foamed concrete

W. H. TAYLOR, *Constructional Review* (Sidney), V. 22, No. 6, Oct. 1949, pp. 11-18

The characteristics of foamed concrete and the cementing materials and aggregates used in making it are considered briefly. For mechanical foaming two methods are described. In the first the foaming agent is dissolved in a part of the mixing water and the resulting foam combined mechanically with the mortar or dry constitutuents. In the second method all the materials are mixed together with compressed air being used in some cases to assist foaming. Under this procedure the foaming agent is dissolved in the mixing water.

To induce foaming chemically, the use of aluminum and zinc powders is described and brief mention is made of calcium carbide or hydrogen peroxide and bleaching powder in this role. The combination of chemical and mechanical foaming also is discussed.

Taking into account the high shrinkage of this material, casting and curing procedures are outlined to produce units of the desired dimensions. Finally, density, strength, dimensional change, thermal conductivity and other properties are considered.

Proportioning concretes for workability

JAMES M. ANTILL, *Journal of the Institution of Engineers*, Australia, Vol. 21, No. 6, June 1949, pp. 89-96 Reviewed by J. R. SHANK

The paper is a general discussion of the design of concrete mixtures in which a principal issue is workability. A number of procedures are discussed; however, those given by Charles T. Kennedy (ACI JOURNAL, Feb. 1940) and Harold Smith (*Civil Engineering and Public Works Review*, Sept. 1946) are given most prominence. Comparisons are made by means of an example of (1) Bolomey's Ideal Hypothesis, (2) U. S. Bureau of Reclamation Method, (3) Fineness Modulus Method, and (4) Patterson's Method, with those of Kennedy and Smith.

A workability test in which the apparatus employed comprises two conical hoppers coaxially located at fixed heights above a standard cylinder mold is described. The procedure is as follows: a measured amount of concrete is loaded into the upper hopper and allowed to drop suddenly into the lower hopper whence it is again dropped into the cylinder mold. The degree of compaction is computed from observations of the amount of concrete overflowing from the mold.

Charts and diagrams showing limits such as the no-slump line and the segregation line, a constant workability line and areas of low and high workability are shown for varying water-cement ratios and sand proportions.

Construction starts on prestressed concrete bridge in Philadelphia

E. R. SCHOFIELD, *Civil Engineering*, V. 19, No. 7, July, 1949, pp. 32-34 Reviewed by J. R. SHANK

The Lincoln Drive bridge at Walnut Lane in Fairmount Park in Philadelphia is made up of prestressed heavy I-shaped concrete girders having a central span of 160 ft and end spans of 74 ft each. Girders are 6 ft 7 in. deep, set side by side, 13 in the central span and 7 in the end spans. All wires are 0.276 in. in diameter. The longitudinal steel of the central span girders consists of four groups of 64 wires each, two groups straight in the bottom flange of the I, one group bent up through the web to half height at the ends, and one up to the top flange. These wires are in sheaths $2\frac{1}{2}$ x 8 in. Cross-binding prestressed wires in groups of 16 each at 14 ft $5\frac{3}{4}$ in. spacing finally bind all girders laterally. These are in 20 in. widths making full rectangular shaped diaphragms which help develop the architectural treatment of the bridge. Extra wide diaphragms are used in the end spans.

The steel of the prestressing wires has an ultimate strength of 210,000 psi and a yield strength of 160,000 psi and works under an estimated maximum stress of 105,000 psi after shrinkage and plastic flow have taken place in the concrete. The concrete has 5400 psi ultimate strength and works under a stress of 1800 psi finally.

Each central span girder, weighing 150 tons, was cast over its final support but not in final position laterally. After the concrete had hardened sufficiently, the wires were stretched to about 125,000 psi and held at the ends by plates in the form of shallow tapered widths working in shallow tapered slots in heavy plates with edges applied to the ends of the girder. Two wires were held by each taper wedge system. After prestressing, grout was forced into the sheaths. The wedge plate assemblies were also grouted in finally.

The design was developed in Belgium by Gustav Magnel.

Reconstruction in Toscany (La ricstruzione nella Toscana)

GIORGIO NEUMANN, *L'Industria italiana del cemento*, Apr. 1948 Reviewed by J. J. POLIVKA

The author describes the reconstruction work performed in reinforced concrete and emphasizes the relatively low vulnerability of this material by bombing, citing the railroad station in Pisa. A huge Fiat automobile plant in Florence, built of reinforced concrete in 1939-1940, was subjected to repeated heavy bombing and considerable but localized damage was caused to individual structural members. The floors of three-story buildings with heavy girders were supported by columns 65 ft on centers in both directions. For wider clearance some of these columns were eliminated thus increasing the span of the girders to 130 ft. Each bomb cut holes in the floors but the heavy girders, 8 x 80 in., were only slightly damaged if not directly hit, and the concrete as a whole was still standing, permitting good repair. Even half-destroyed girders, some of them with considerably reduced depth were easily repaired due to properly placed heavy reinforcement. The reconstruction proceeded in the following way which proved to be most effective also in other similar cases: the relatively narrow girders were widened to about 12 in. which was permissible on account of the considerable depth—6 ft 6 in.—and a perfect anchorage was provided for intermediate beams and slab. Special care was required in drilling holes for dowels and anchor bars to avoid cutting of tensile bars. The most difficult cases were repairs of damage caused by mines. Usually the mines were placed at the most important points where the destruction of a structural member is expected to cause the collapse of a larger part of the building, such as a corner column or pilaster.

Another important field of reconstruction is bridges. In the city of Florence alone, five bridges were destroyed but up to date only one, the Victory bridge, has been rebuilt. More progress in reconstruction was made with railroads. Nearly all bridges originally built in stone masonry and in steel were rebuilt in reinforced concrete because of greater durability and practically no maintenance. Noteworthy is the new continuous arch bridge over the Arno River between Carmignano and Montelupo.

Structural design and calculation of the cement mill at Stora Vika (Byggnadskonstruktionerna vid cementfabriken i Stora Vika)

STIG LAGERKRANZ, *Betong* (Stockholm), V. 34, No. 3, 1949, pp. 149-169 AUTHOR'S SUMMARY

This paper describes the design and construction of various structures forming part of the new cement mill at Stora Vika built by Skanska Cementaktiebolaget.

The structures erected for the ropeway comprise a loading station, 5 concrete supporting poles, road protective structures, and an unloading station which serves also as a tensioning station for the carrier ropes. The tensile force of 35 tons per rope is pro-

duced by a concrete counterweight attached to each rope. The total overturning moment acting on the structure is about 2400 metric tons. The building was constructed by means of a sliding form. While the form was moved, the necessary openings were made for the intermediate floor structures, which were cast at a later stage, as well as for windows, doors, etc.

The kiln building comprises an open bay, about 185 m in length, housing two rotary kilns, and a part, about 118 m in length, containing the clay stores, the pug mills, and the flotation plant. The building is provided with an expansion joint running completely through it. The load-bearing structure of the roof consists of three-hinged arches made of prestressed concrete beams and tie rods consisting of 4 round iron bars, 1 in. in diameter, fastened together without joints. The ridge pieces are made of concrete beams, 12 x 16 in. in cross section, with prestressed reinforcement in the hottest section of the kiln house, and of 3 x 8-in. timbers in the other parts of the building. The roof is covered with corrugated Eternite.

Each of the rotary kilns, about 150 m in length, is supported on 6 foundation blocks which were designed to withstand horizontal forces in the longitudinal and transverse direction of the kiln not exceeding 100 tons per kiln per block. The motors and other accessories for driving the kilns are carried on simply supported floor structures. The gear pressures, which amount to 33 tons per kiln in normal operation, are sustained by bending and torsion of the adjacent kiln foundations. The soil characteristics on the site vary widely, and the foundations are carried on piles, gravel, sand mixed with moraine, and on rock.

The slurry basin is 32.5 m in diameter and 6.5 m in height. The diameter of the superstructure is 35.0 m. The pressure exerted by the liquid on the walls of the basin does not produce any horizontal stresses, and is sustained by 20 radially arranged supporting trestles between which the wall of the basin is made of continuous concrete slabs reinforced in two directions. The roof structure consists of a concrete dome 8 cm thick.